CHAPTER 17

BANK PROTECTION

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17.1 INTRODUCTION

17.1.1 Purpose

One of the hazards of placing a highway near a river or stream channel or other water body is the potential for erosion of the highway embankment by moving water. If erosion of the highway embankment is to be prevented, bank protection must be anticipated, and the proper type and amount of protection must be provided in the right locations.

Four methods of protecting a highway embankment from bank erosion are available to the designer. These are:

- relocating the highway away from the stream or water body,
- moving the water body away from the highway (channel change),
- changing the direction of the current with training works, and
- protecting the embankment from erosion.

This Chapter provides procedures for designing revetments to be used as channel bank protection and channel linings on streams having design discharges greater than 50 ft^3/s. Procedures also are presented for riprap protection at bridge piers and abutments. For smaller discharges, the procedures presented in the Channels Chapter should be used. Emphasis in this Chapter is placed on rock riprap revetments due to their low cost, environmental considerations, flexible characteristics and widespread acceptance. Other channel stabilization methods (e.g., spurs, guide banks, retard structures, longitudinal dikes, bulkheads) are discussed in HEC 20 *(7)*.

17.1.2 Erosion Potential

Channel and bank stabilization is essential to the design of any structure affected by the water environment. The identification of the potential for bank erosion, and the subsequent need for stabilization, is best accomplished through observation. A three-level analytical procedure is provided in HEC 20 *(7)*. That procedure is described in this *Manual* in the Channels Chapter. The three-level analysis provides a rigorous procedure for determining the geomorphological characteristics, evaluation of existing conditions through field observations, and determining the hydraulic and sediment transport properties of the stream. If sufficient information is obtained at any level of the analysis to solve the problem, the procedure may be stopped without proceeding on to the succeeding levels.

Observations provide the most positive indication of erosion potential. Observation comparison can be based on historic information or current site conditions. Aerial photographs, old maps, survey notes, bridge design files and river survey data (available at UDOT and at Federal agencies) and gaging station records. Interviews of local residents can provide documentation of recent channel movements or bank instabilities.

Current site conditions can be used to evaluate stability. Even when historic information indicates that a bank has been relatively stable in the past, local conditions may indicate more recent instabilities. Local site conditions that are indicative of instabilities may include:

- tipping and falling vegetation along the bank,
- cracks along the bank surface,

- the presence of slump blocks,
- fresh vegetation lying in the channel near the channel banks,
- deflection of channel flows in the direction of the bank due to recently deposited obstructions or channel course changes,
- fresh vertical face cuts along the bank,
- locally high velocities along the bank,
- new bar formation downstream from an eroding bank,
- local headcuts, and
- pending or recent cutoffs.

It is important to recognize that the presence of one of these conditions alone does not, by itself, indicate an erosion problem; some bank erosion is common in all channels even when the channel is stable.

17.1.3 Symbols and Definitions

To provide consistency within this Chapter and throughout this *Manual* the symbols in Table 17-1 will be used. These symbols were selected because of their wide use in many bank and shore protection publications. Where the same symbol is used for more than one definition, the symbol will be defined where it is used.

17.2 POLICY

Highway alignments and improvements often cross, encroach upon or otherwise require construction of a new channel or modification of an existing channel. It is necessary to protect the public, the highway investment, and the environment from the natural reaction of a stream to the highway changes. The designer shall design highway facilities to preserve natural drainage conditions to the maximum extent possible, thus protecting and maintaining the environment.

TABLE 17-1 — Symbols and Definitions

Symbol	Definition	Units
AOS	Apparent opening size in filter cloth	in
A	Coefficient used to determine the apparent opening size	
C	Coefficient; relates free vortex motion to velocity streamlines for unequal radius or curvature	-
C_u	Uniformity coefficient	
D_{50}	The median bed material size	ft or in
D_{15}	The 15% finer particle size	ft or in
D_{85}	The 85% finer particle size	ft or in
d_{avg}	Average flow depth in the main flow channel	ft
D_s	Estimated probable maximum depth of scour	ft
g	Gravitational acceleration (32.2 ft/s^2)	ft/s^2
H	Wave height	ft
k	Permeability	ft/s or in/s
K_1	Correction term reflecting bank angle	-
n	Manning's roughness coefficient	-
O_{95}	Opening size in the geotextile material for which 95% of the openings are smaller	in
Q_{mc}	Discharge in the zone of main channel flow	ft^3/s
R	Hydraulic radius	ft
R	Wave runup	ft
R_o	Mean radius of the channel centerline at the bend	ft
S_f, S	Friction slope or energy grade line slope	ft/ft
SF	Stability factor	-
S_s, s	Specific gravity of the riprap	-
T	Top width of the channel between its banks	ft
V	Velocity	ft/s
V_a	Average channel velocity	ft/s
W_{50}	Weight of the median particle	lb
Z	Superelevation of the water surface	ft
γ	Unit weight of the riprap material	lb/ft^3
θ	Bank angle with the horizontal	degrees
ϕ	Riprap material's angle of repose	degrees

17.3 DESIGN CRITERIA

To provide an acceptable standard level of service, the Department traditionally employs established design storms that are based on the importance of the transportation facility to the system and the allowable risk for that facility. Selection of the appropriate design condition from these standards is a matter of professional judgment and recognition that design flow standards represent an engineering consensus on reasonable values. Although it is rarely either possible or practical to provide protection for the greatest probable flow, actual design must consider the consequences of greater events that may produce more severe hydraulic conditions. Under certain conditions, it may be appropriate to establish the level of risk allowable for a site and to design to that level. In addition, design standards of other agencies that have control or jurisdiction over the waterway or facility concerned should be incorporated or addressed in the design. Specific design criteria recommendations are summarized below and are presented in greater detail in Section 17-6.

Minimum Criteria	Frequency
Design Flow Freeboard at Design Flow	10- to 100-yr flood 3 ft - 2 ft
Check Flow Freeboard at Check Flow	25- to 500-yr flood 1 ft – no freeboard

17.4 BANK AND LINING FAILURE MODES

17.4.1 Introduction

Prior to designing a bank stabilization scheme, it is important to be aware of common erosion mechanisms, revetment failure modes and the causes or driving forces behind bank erosion processes. Inadequate recognition of potential erosion processes at a particular site may lead to failure of the revetment system.

Many causes of bank erosion and revetment failure have been identified. Some of the common causes include abrasion, debris flows, water flow, eddy action, flow acceleration, unsteady flow, freeze/thaw, human actions on the bank, ice, precipitation, waves, toe erosion and subsurface flows. It is most often a combination of causes that produce bank and revetment failures, and the primary mechanism or cause is usually difficult to determine. Failures may be classified by failure mode including:

- particle erosion,
- translational slide,
- modified slump, and
- slump.

For more detail, see HEC 11 *(3)*.

17.4.2 Particle Erosion

Particle erosion is the most commonly considered erosion mechanism. Particle erosion results where the tractive force exerted by the flowing water exceeds the bank material's ability to resist movement. In addition, if displaced stones are not transported from the eroded area, a mound of displaced rock will develop on the channel bed. This mound has been observed to cause flow concentration along the bank, resulting in further bank erosion.

A special type of particle erosion results in loss of the underlying material resulting in undermining and eventual collapse of the revetment protection. Usually, the underlying material is lost through the revetment or piped under the toe of the revetment protection. This failure is very common in, and extremely damaging to, rigid types of protective linings. Providing a sufficient toe-down or providing a suitable filter, either natural or fabric in conjunction with hydrostatic relief features, will prevent this failure.

Another frequent type of particle erosion failure occurs at the edges of the protective feature. The interface creates turbulence which, in turn, increases the local stress placed on the protective layer, underlying layer and the natural bank material beyond the revetment. This failure area needs to receive special attention because extension of the protective feature usually only moves, but does not eliminate, the failure.

17.4.3 Translation Slide

A translational slide is a bank failure caused by the downslope movement of a mass of stones, with the fault line on a horizontal plane. The initial phases of a translational slide are indicated by cracks in the upper part of the riprap bank that extend parallel to the channel. As the slide progresses, the lower part of riprap separates from the upper part and moves downslope as a homogeneous body. A resulting bulge may appear at the base of the bank if the channel bed is not scoured.

17.4.4 Modified Slump

The bank failure referred to as modified slump is the mass movement of material along an internal slip surface within the riprap blanket; the underlying material supporting the riprap does not fail. This type of failure is similar in many respects to the translational slide, but the geometry of the damaged riprap is similar in shape to initial stages of failure caused by particle erosion.

17.4.5 Slump

Slump is a rotational-gravitational movement of material along a surface of rupture that has a concave upward curve. The cause of slump failures is related to shear failure of the underlying base material that supports the riprap revetment. The primary feature of a slump failure is the localized displacement of base material along a slip surface, which is usually caused by excess pore pressure that reduces friction along a fault line in the base material.

17.5 REVETMENT TYPES

17.5.1 Common Types

The types of slope protection or revetment commonly used for bank and shore protection and stabilization include:

- rock and rubble riprap,
- wire-enclosed rock,
- precast concrete block,
- grouted rock,
- concrete slope protection,
- grouted fabric slope pavement,
- sand/cement bags, and
- soil cement.

Descriptions of each of these revetment types are included in this Section. There are two relatively new types of bank protection methods. One is bendway weirs, which are discussed in Section 17.7.9, and the other is vegetative plantings and soil bioengineering systems, which are discussed in Section 17.7.10. As the name implies, bending weirs are applicable to channel bends, and bioengineering systems are more applicable to stream relocations.

17.5.2 Rock and Rubble Riprap

Riprap has been described as a layer or facing of rock, dumped or hand-placed to prevent erosion, scour or sloughing of a structure or embankment. Materials other than rock are also referred to as riprap; these include rubble, broken concrete slabs and preformed concrete shapes (slabs, blocks, rectangular prisms.). These materials are similar to rock in that they can be hand placed or dumped onto an embankment to form a flexible revetment.

17.5.3 Wire-Enclosed Rock

Wire-enclosed rock, or gabion, revetments consist of rectangular wire-mesh baskets filled with rock. These revetments are formed by filling pre-assembled wire baskets with rock and anchoring them to the channel bottom or bank. Wire-enclosed rock revetments are generally of two types distinguished by shape — mattresses and blocks. In mattress designs, the individual wire mesh units are laid end-to-end and side-to-side to form a mattress layer on the channel bed or bank. The gabion baskets comprising the mattress generally have a depth dimension that is much smaller than their width or length. Block gabions, in contrast, are more equal-dimensional, having depths that are approximately the same as their widths, and of the same order of magnitude as their lengths. They are typically rectangular or trapezoidal in shape. Block gabion revetments are formed by stacking the individual gabion blocks in a stepped fashion.

17.5.4 Precast Concrete Block

Precast concrete block revetments consist of preformed sections which are butted together or joined in some fashion to form a continuous blanket or mat. The concrete blocks, which make up the mats, differ in shape and method of articulation but share certain common features. These features include flexibility, rapid installation and provisions for establishment of vegetation within the revetment. The permeable nature of these revetments permits free draining of the bank materials; the flexibility, although limited, allows the mattress to conform to minor changes in the bank geometry. Their limited flexibility, however, makes them subject to undermining in environments characterized by large and relatively rapid fluctuations in the surface elevation of the channel bed and/or bank. Unlike wire-enclosed rock, the open nature of the precast concrete blocks does promote growth of volunteer vegetation within the revetment.

17.5.5 Grouted Rock

Grouted rock revetment consists of rock slope protection having voids filled with concrete grout to form a monolithic armor. Grouted rock is a rigid revetment; it will not conform to changes in the bank geometry due to settlement. As with other monolithic revetments, grouted rock is particularly susceptible to failure from undermining and the subsequent loss of the supporting bank material. Although it is rigid, grouted rock is not strong; therefore, the loss of even a small area of bank support can cause failure of large portions of the revetment.

An alternative to grouted rock is partially grouted rock. In general, the objective of partially grouted rock is to increase the stability of the riprap without sacrificing all of the flexibility. As with fully grouted rock, partially grouted rock may be well suited for areas where rock of sufficient size is not available to construct a loose riprap revetment. The grout for partially grouted rock is placed on the riprap leaving significant voids in the riprap matrix and considerable open space on the surface. The holes in the grout allow for drainage of pore water so a filter is required. The grout forms conglomerates of rock so that the stability against particle erosion is greatly improved and, as with fully grouted riprap, a smaller thickness of stone can be used. Although not as flexible as riprap, partially grouted rock will conform somewhat to bank settlement and toe exposure.

17.5.6 Concrete Slope Protection

Concrete pavement revetments (concrete slope pavement) are cast in place on a prepared slope to provide the necessary bank protection. Like grouted rock, concrete pavement is a rigid revetment that does not conform to changes in bank geometry due to a removal of foundation support by subsidence, undermining, outward displacement by hydrostatic pressure, slide action, or erosion of the supporting embankment at its ends. The loss of even small sections of the supporting embankment can cause complete failure of the revetment system. Concrete pavement revetments are also among the most expensive streambank protection designs. In the past, concrete pavement has been best utilized as a subaqueous revetment (on the bank below the water surface) with vegetation or some other less expensive upper-bank treatment.

17.5.7 Grouted Fabric Slope Pavement

Grouted fabric slope pavement revetments are constructed by injecting sand-cement mortar between two layers of double-woven fabric that has first been positioned on the slope to be protected. Mortar may be injected into this fabric envelope either underwater or in the dry. The fabric enclosure prevents dilution of the mortar during placement underwater. The two layers of fabric act first as the top and bottom form to hold the mortar in place while it hardens. This fabric, to which the mortar remains tightly bonded, then acts as tensile reinforcing to hold the mortar in place on the slope. These revetments are analogous to slope paving with reinforced concrete. The bottom layer of fabric acts as a filter cloth underlayment to prevent loss of soil particles through any cracks that may develop in the revetment as a result of soil subsidence. Often, greater relief of hydrostatic uplift is provided by weep holes or filter points that are normally woven into the fabric and remain unobstructed by mortar during the filling operation. One advantage to this type of revetment protection is that the concrete can be pumped into tight locations where there is little room for equipment such as under existing bridges to protect the abutments.

17.5.8 Sand-Cement Bags

Sand-cement bags generally consist of a dry mix of sand and cement placed in a burlap or other suitable bag. They are hand placed in contact with adjacent bags. They require firm support from the protected bank. Usually a filter fabric is placed underneath this type of riprap. Adequate protection of the terminals and toe is essential. The riprap has little flexibility and low tensile strength and is susceptible to damage particularly on flatter slopes where the area of contact between the bags is less.

17.5.9 Soil-Cement

Soil-cement generally consists of a dry mix of sand and cement and admixtures batched in a central mixing plant. It is usually transported, placed by equipment capable of producing the width and thickness required and compacted to the required density. Control of the moisture and time after introduction of the mixing water is critical. Curing is required. This results in a rigid protection. Soil-cement can be placed either as a lining or in stepped horizontal layers. The stepped horizontal layers are extremely stable, provided that toe scour protection has been incorporated into the design.

17.6 DESIGN CONCEPTS

17.6.1 Introduction

Design concepts related to the design of bank protection are discussed in this Section. Subjects covered in this Section include design discharge, flow types, section geometry, flow in channel bends, flow resistance and extent of protection.

17.6.2 Design Discharge

Design flow rates for the design or analysis of highway structures in the vicinity of rivers and streams usually have a 10- to 50-yr recurrence interval. In most cases, these discharge levels will be applicable to the design of revetment systems; however, the hydraulics engineer should be aware that, in some instances, a lower discharge might produce hydraulically worse conditions with respect to revetment stability. It is suggested that several discharge levels be evaluated to ensure that the design is adequate for all discharge conditions up to that selected as the design discharge for structures associated with the revetment type.

17.6.3 Flow Types

Open channel flow can be classified from three points of reference. These are:

- uniform, gradually varying or rapidly varying flow;
- steady or unsteady flow; and
- subcritical or supercritical flow.

Design relationships presented in this Chapter are based on the assumption of uniform, steady, subcritical flow. These relationships are also valid for gradually varying flow conditions. Although the individual hydraulic relationships presented are not in themselves applicable to rapidly varying, unsteady or supercritical flow conditions, procedures are presented for extending their use to these flow conditions (see the Channels Chapter for more details related to channel design).

Rapidly varying, unsteady flow conditions are common in areas of flow expansion, flow contraction and reverse flow. These conditions are common at and immediately downstream of bridges. Supercritical or near supercritical flow conditions are common at bridge constrictions and culvert outlets on steep-sloped channels.

Non-uniform, unsteady and near supercritical flow conditions create stresses on the channel boundary that are significantly different from those induced by uniform, steady, subcritical flow. These stresses are difficult to assess quantitatively. The stability factor method of riprap design presented in Section 17.7.1 provides a means of adjusting the final riprap design (that is based on relationships derived for steady, uniform, subcritical flow) for the uncertainties associated with these other flow conditions. The adjustment is made through the assignment of a stability factor. The magnitude of the stability factor is based on the level of uncertainty inherent in the design flow conditions.

17.6.4 Section Geometry

Design procedures presented in this Chapter require knowing the channel cross section geometry. The cross section geometry is necessary to establish the hydraulic design parameters (e.g., flow depth, topwidth, velocity, hydraulic radius) required by the riprap design procedures and to establish a construction cross section for placement of the revetment material. When the entire channel perimeter is to be stabilized, the selection of an appropriate channel geometry is only a function of the desired channel conveyance properties and any limiting geometric constraints. However, when the channel bank alone is to be protected, the design must consider the existing channel bottom geometry.

The development of an appropriate channel section for analysis is very subjective. The intent is to develop a section that reasonably simulates a worst-case condition with respect to riprap stability. Information that can be used to evaluate channel geometry includes current channel surveys, past channel surveys (if available) and current and past aerial photos. The effect channel stabilization will have on the local channel section must be considered.

To establish an existing channel's geometrics, it is necessary to survey the channel bottom profile and a sufficient number of cross sections to adequately describe the channel. In addition to current channel surveys, historic surveys can provide valuable information. A comparison of current and past channel surveys at the project location provides information on the general stability of the site and a history of past channel geometry changes. Often, past surveys at a particular site will not be available. If this is the case, past surveys at other sites in the vicinity of the design location may be used to evaluate past changes in channel geometry.

17.6.5 Flow In Channel Bends

Flow conditions in channel bends are complicated by the distortion of flow patterns in the vicinity of the bend. In long, relatively straight channels, flow conditions are uniform and symmetrical about the centerline of the channel; however, in channel bends, the centrifugal forces produce secondary currents that cause non-uniform and non-symmetrical flow conditions.

Special consideration must be given to the increased velocities and shear stresses that are generated as a result of non-uniform flow in bends.

Superelevation of flow in channel bends is another important consideration in the design of revetments. Although the magnitude of superelevation is generally small when compared with the overall flow depth in the bend (usually less than 1 ft), it should be considered when establishing freeboard limits for bank protection schemes on sharp bends. The magnitude of superelevation at a channel bend may be estimated for subcritical flow by the following Equation:

$$Z = [(V_a^2 T)/(gR_o)] \tag{17.1}$$

where:
- Z = superelevation of the water surface, ft
- V_a = mean channel velocity, ft/s
- T = water surface width at section, ft
- g = gravitational acceleration, 32.2 ft/s^2
- R_o = the mean radius of the channel centerline at the bend, ft

17.6.6 Flow Resistance

The hydraulic analysis performed as a part of the revetment design process requires the estimation of Manning's roughness coefficient. Physical characteristics upon which the resistance equations are based include the channel base material, surface irregularities, variations in section geometry, bed form, obstructions, vegetation, channel meandering, flow depth and channel slope. In addition, seasonal changes in these factors must also be considered. See the Channels Chapter for a discussion of the selection of Manning's n values.

17.6.7 Extent of Protection

The extent of protection refers to the longitudinal and vertical extent of protection required to adequately protect the channel bank.

17.6.7.1 Longitudinal Extent

The longitudinal extent of protection required for a particular bank protection scheme is highly dependent on local site conditions. In general, the revetment should be continuous for a distance greater than the length that is impacted by channel flow forces severe enough to cause dislodging and/or transport of bank material. Although this is a vague criterion, it demands serious consideration. Review of existing bank protection sites has revealed that a common error in stream bank protection is to provide protection too far upstream and not far enough downstream.

One criterion for establishing the longitudinal limits of protection required is illustrated in Figure 17-1. As illustrated, the minimum distances recommended for bank protection are an upstream distance of 1.0 channel width and a downstream distance of 1.5 channel widths from corresponding reference lines. All reference lines pass through tangents to the bend at the bend entrance or exit. This criterion is based on an analysis of flow conditions in symmetric channel bends under ideal laboratory conditions. Real-world conditions are rarely as simplistic.

In actuality, many site-specific factors have a bearing on the actual length of bank that should be protected. A designer will find the above criteria difficult to apply on mildly curving bends or on channels having irregular, non-symmetric bends. Also, other channel controls (e.g., bridge abutments) might already be producing a stabilizing effect on the bend so that only a part of the channel bend needs to be stabilized. The magnitude or nature of the design flow event might cause erosion problems only in a very localized portion of the bend, requiring that only a short channel length be stabilized; therefore, the above criteria should be used only as a starting point. Additional analysis of site-specific factors is necessary to define the actual extent of protection required.

Field reconnaissance is a useful tool for the evaluation of the longitudinal extent of protection required, particularly if the channel is actively eroding. In straight channel reaches, scars on the channel bank may be useful to help identify the limits required for channel bank protection. In this case, it is recommended that upstream and downstream limits of the protection scheme be extended a minimum of one channel width beyond the observed erosion limits.

In curved channel reaches, the scars on the channel bank can be used to establish the upstream limit of erosion. Here, a minimum of one channel width should be added to the observed upstream limit to define the limit of protection. The downstream limit of protection required in curved channel reaches is not as easy to define. Because the natural progression of bank erosion is in the downstream direction, the present visual limit of erosion might not define the ultimate downstream limit. Additional analysis based on consideration of flow patterns in the channel bend may be required.

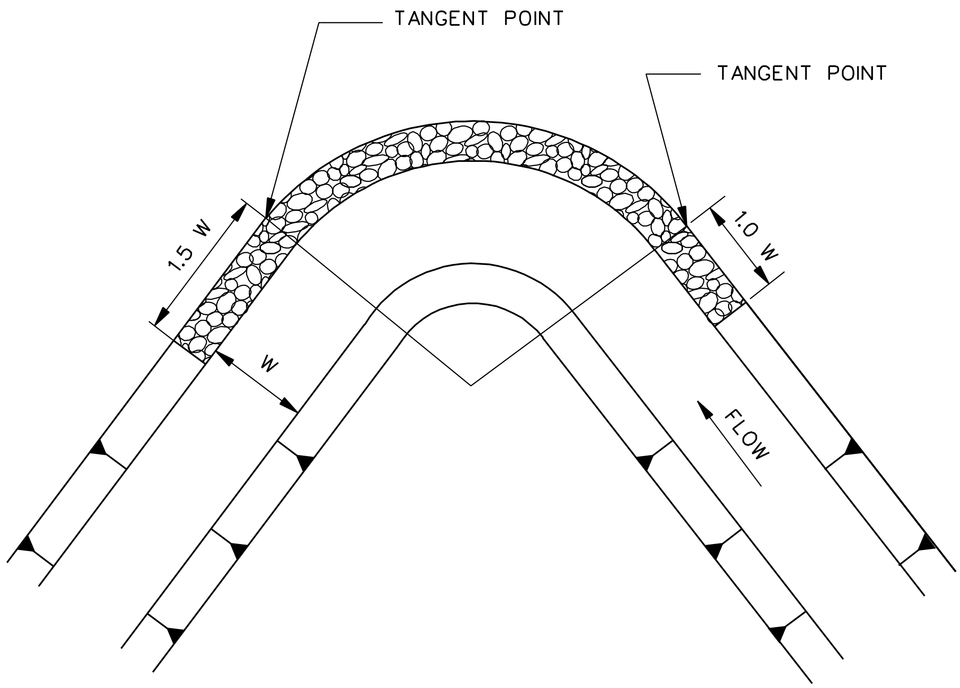


FIGURE 17-1 — Longitudinal Extent of Revetment Protection

17.6.7.2 Vertical Extent

The vertical extent of protection required of a revetment includes design height and foundation or toe depth.

17.6.7.2.1 <u>Design Height</u>

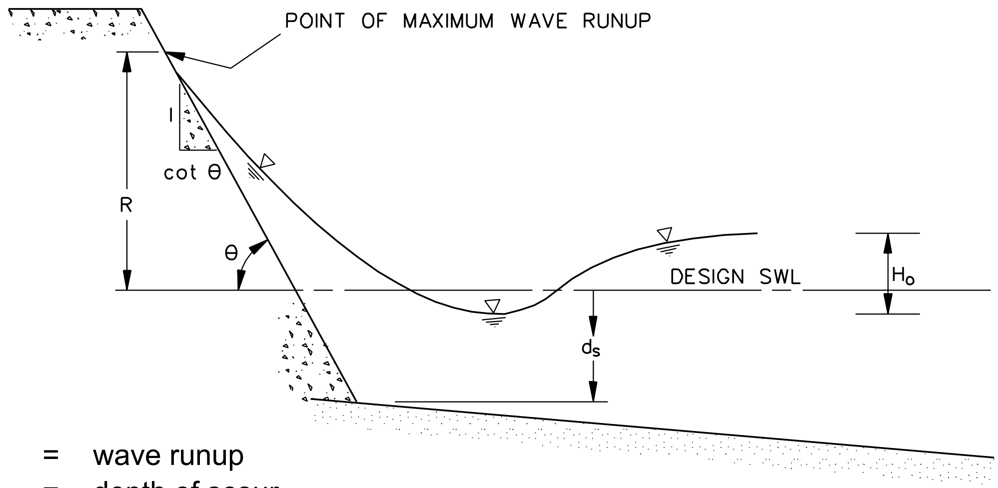
The design height of a revetment should be equal to the design high-water elevation plus some allowance for freeboard. Freeboard is provided to ensure that the desired degree of protection will not be compromised by such factors as:

- wave action (from wind or boat traffic);
- superelevation in channel bends;
- hydraulic jumps; and
- flow irregularities due to piers, transitions and flow junctions.

In addition, erratic phenomena (e.g., unforeseen embankment settlement, the accumulation of silt, trash and debris in the channel, aquatic or other growth in the channels and ice flows) should be considered when setting freeboard heights. Also, wave run-up on the bank must be considered.

The prediction of wave heights from wind- and boat-generated waves is not as straightforward as other wave sources. Figure 17-2 provides a definition sketch for the wave height discussion to follow. The height of boat-generated waves must be estimated from observations. The height of wind-generated waves is discussed in the Coastal Zone Chapter.

Wave runup is a function of the wave height, wave period, bank angle and the bank surface characteristics (as represented by different revetment materials). For wave heights less than



Key: R = wave runup
d_s = depth of scour
H_O = wave height
SWL = surface water level
θ = bank angle with the horizontal

FIGURE 17-2 — Wave Height Definition Sketch

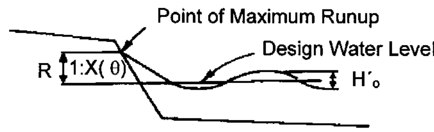
2 ft, wave runup can be computed using Figure 17-3 and Table 17-2. The runup height (R) given in Figure 17-3 is for concrete pavement. Correction factors are provided in Table 17-2 for reducing the runup magnitude for other revetment materials. The correction factor is multiplied times the wave height to get the resulting wave runup (R).

As indicated, there are many factors that must be considered in the selection of an appropriate freeboard height. At a minimum, it is recommended that a freeboard height of 1 ft to 2 ft be used in un-constricted reaches and 2 ft to 3 ft in constricted reaches. FEMA requires 3 ft for levee protection and 4 ft at bridges for the 100-yr flood. When computational procedures indicate that additional freeboard may be required, the greater height should be used. In addition, it is recommended that the designer observe wave and flow conditions during various seasons of the year (if possible), consult existing records and interview persons who have knowledge of past conditions when establishing the necessary vertical extent of protection for a particular revetment installation.

17.6.7.2.2 Toe Depth

Undermining the revetment toe is one of the primary mechanisms of revetment failure. In the design of bank protection, estimates of the depth of scour are needed so that the protective layer is placed sufficiently low in the streambed to prevent undermining. The ultimate depth of scour must consider channel degradation and natural scour and fill processes.

The relationships presented in Equations 17.2 and 17.3 *(1)* can be used to estimate the probable maximum depth of scour due to natural scour and fill phenomenon in straight channels and in channels having mild bends. In application, the depth of scour, d_s, should be measured from the lowest elevation in the cross section. It should be assumed that the low point in the cross section may eventually move adjacent to the protection (even if this is not the case in the current survey):



R = Wave Run Up Height (ft)
H´o = Wave Height (ft)
θ = Bank Angle with the Horizontal

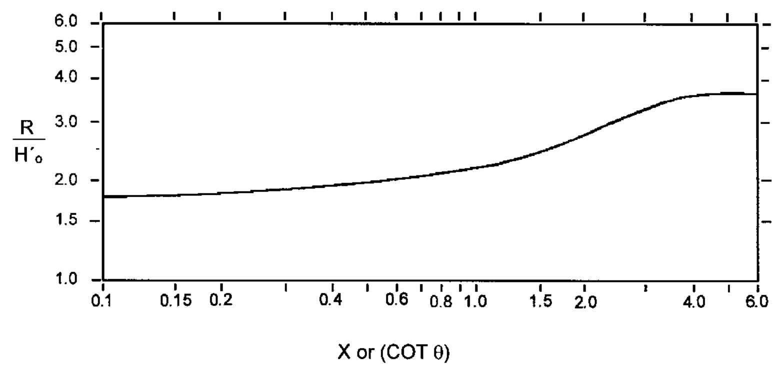


FIGURE 17-3 — Wave Run-Up on Smooth, Impermeable Slopes

TABLE 17-2 — Correction Factors For Wave Run-Up

Slope Surface Characteristics	Placement Method	Correction Factor
Concrete pavement	—	1.00
Concrete blocks (voids < 20%)	fitted	0.90
Concrete blocks (20% < voids < 40%)	fitted	0.70
Concrete blocks (40% < voids < 60%)	fitted	0.50
Grass	—	0.85 – 0.90
Rock riprap (angular)	random	0.60
Rock riprap (round)	random	0.70
Rock riprap (hand-placed or keyed)	keyed	0.80
Grouted rock	—	0.90
Wire-enclosed rocks/gabions	—	0.80

$$d_s = 12 \text{ ft} \quad \text{for } D_{50} < 0.005 \text{ ft} \tag{17.2}$$

$$d_s = 6.5\, D_{50}^{-0.11} \quad \text{for } D_{50} \geq 0.005 \text{ ft} \tag{17.3}$$

where: d_s = estimated probable maximum depth of scour, ft
D_{50} = median diameter of bed material, ft

The depth of scour predicted by Equations 17.2 and 17.3 (see HEC 11 *(3))* must be added to the magnitude of predicted degradation and local scour (if any) to arrive at the total required toe depth.

17.7 DESIGN GUIDELINES

17.7.1 Rock Riprap

This Section contains design guidelines for the design of rock riprap. Guidelines are provided for bank slope, rock size, rock gradation, riprap layer thickness, filter design, edge treatment and construction considerations. In addition, typical construction details are illustrated. In most cases, the guidelines presented apply equally to rock and rubble riprap. Concrete rubble may be used as riprap, provided that it is clear of asphalt and that any protruding reinforcing steel is cut off flush with the surface.

17.7.1.1 Bank Slope

A primary consideration in the design of stable riprap bank protection schemes is the slope of the channel bank. For riprap installations, normally the maximum recommended face slope is 1V:2H. Although it is generally not recommended, the steepest slope acceptable for rubble revetment is 1V:1.5H. To be stable for a given wave or tractive force, a rubble revetment with a steep slope will need larger rubble sizes and greater thicknesses than one with a flatter slope.

17.7.1.2 Rock Size

The stability of a particular riprap particle depends, in part, on its size, expressed either in terms of its weight or equivalent diameter. In the following Sections, relationships are presented for evaluating the riprap size required to resist particle and wave erosion forces.

17.7.1.2.1 Resistance to Particle Erosion

Two methods or approaches have been used historically to evaluate a material's resistance to particle erosion. These methods are the permissible velocity approach and the permissible tractive force (shear stress) approach. Under the permissible velocity approach, the channel is assumed stable if the computed mean velocity is lower than the maximum permissible velocity. The tractive force (boundary shear stress) approach focuses on stresses developed at the interface between flowing water and materials forming the channel boundary.

17.7.1.2.2 Design Relationship

A riprap design relationship that is based on tractive force theory, yet has velocity as its primary design parameter, is presented in Equation 17.4. The design relationship in Equation 17.4 is based on the assumption of uniform or gradually varying flow. Figure 17-4 presents a graphical solution to Equation 17.4. Equation 17.5 can be solved using Figures 17-5 and 17-6:

$$D_{50} = 0.001\ V_a^3 / (d_{avg}^{0.5}\ K_1^{1.5}) \qquad (17.4)$$

where:
- D_{50} = the median riprap particle size, ft
- C = $C_{sg}\ C_{sf}$ = correction factor (described below)
- V_a = the average velocity in the main channel, ft/s
- d_{avg} = the average flow depth in the main flow channel, ft

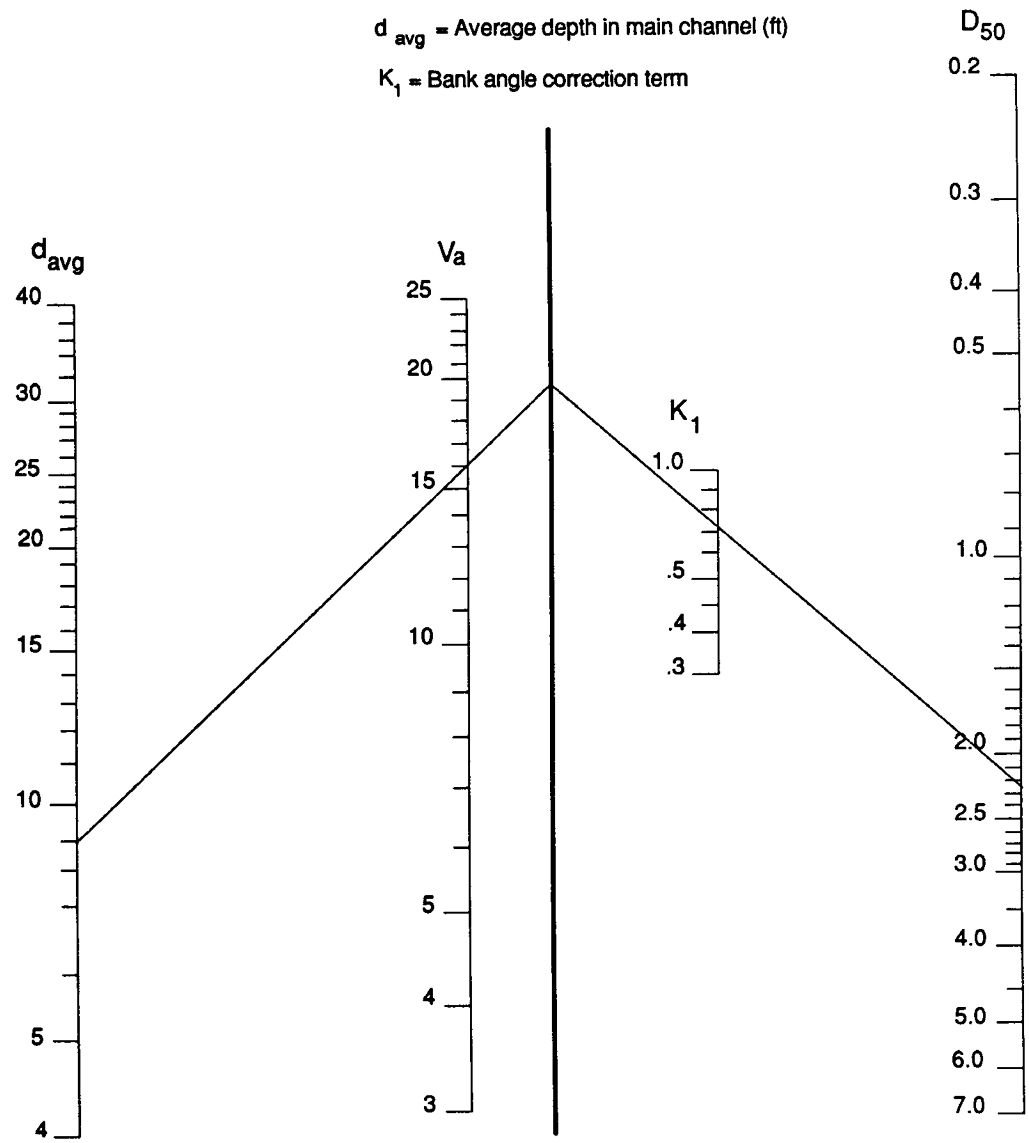
$D_{50} = 0.001\, V_a^3 / (d_{avg}^{1/2} K_1^{3/2})$

D_{50} = Median Riprap Size (ft.)

V_a = Average velocity in main channel (ft/sec)

d_{avg} = Average depth in main channel (ft)

K_1 = Bank angle correction term



Example

Given:
- V_a = 16 ft/sec
- d_{avg} = 9 ft
- K_1 = 0.72

Find:
- D_{50}

Solution:
- D_{50} = 2.25

FIGURE 17-4 — Riprap Size Relationship *(3)*

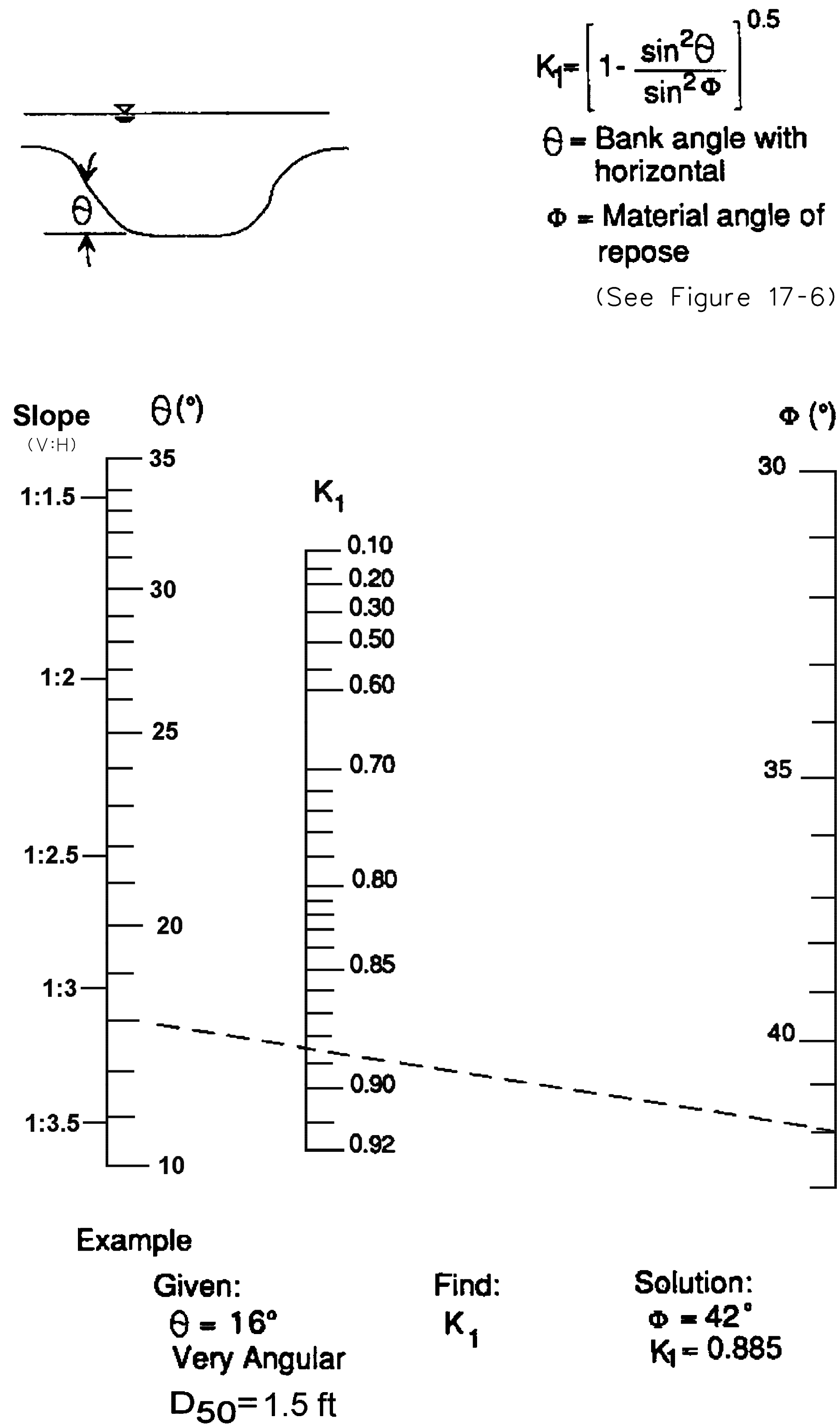


FIGURE 17-5 — Bank Angle Correction Factor (K_1) Nomograph

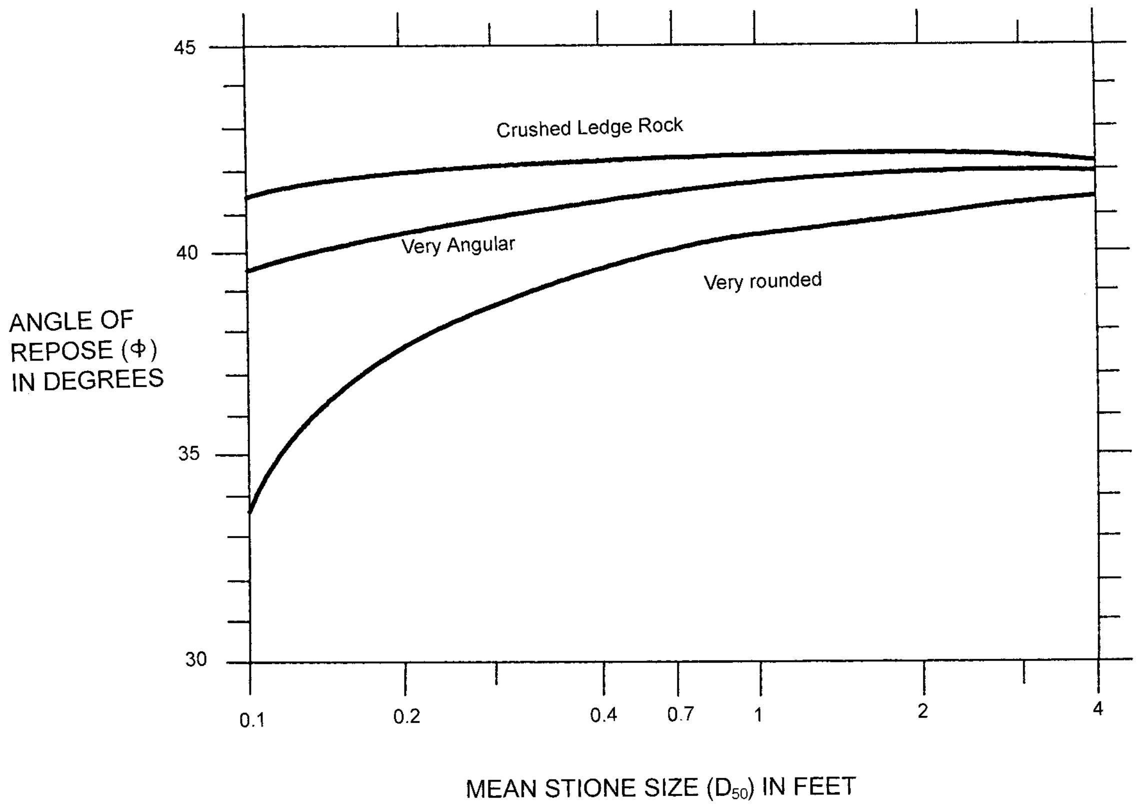


FIGURE 17-6 — Angle of Repose of Riprap in Terms of Mean Size and Shape of Stone

K_1 is defined as:

$$K_1 = [1 - (\sin^2\theta/\sin^2\phi)]^{0.5} \quad (17.5)$$

where: θ = the bank angle with the horizontal

ϕ = the riprap material's angle of repose

The average flow depth and velocity used in Equation 17.4 are main channel values. The main channel is defined as the area between the channel banks (see Figure 17-7).

Equation 17.4 is based on a rock riprap specific gravity of 2.65 and a stability factor of 1.2. The stability factor (SF) is defined as the ratio of the average tractive force exerted by the flow field and the riprap material's critical shear stress. If the stability factor is greater than 1, the critical shear stress of the material is greater than the flow-induced tractive stress, and the riprap is considered to be stable. Equations 17.6 and 17.7 present correction factors for other specific gravities and stability factors:

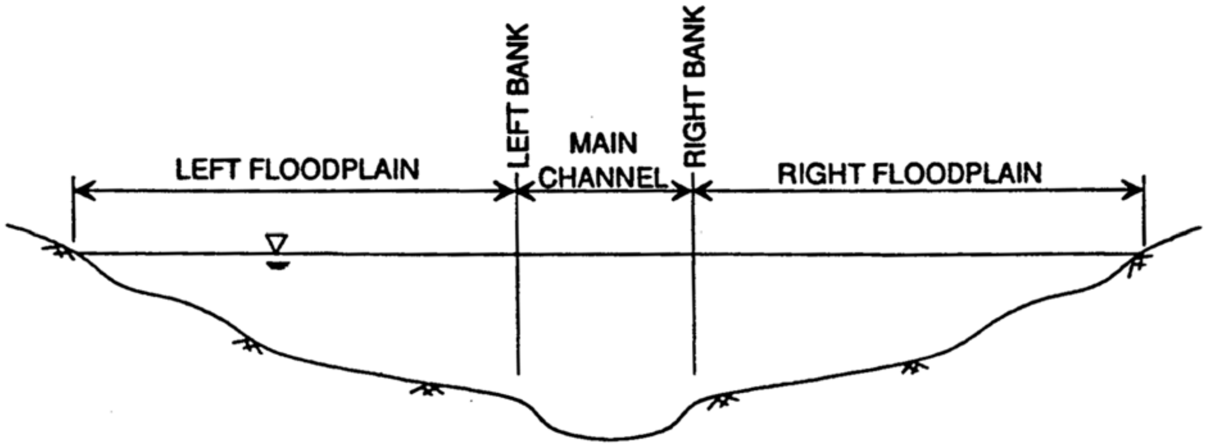


FIGURE 17-7 — Definition Sketch; Channel Flow Distribution

$C_{sg} = 2.12/(S_s - 1)^{1.5}$ (17.6)

where: S_s = the specific gravity of the rock riprap

$C_{sf} = (SF/1.2)^{1.5}$ (17.7)

where: SF = the stability factor to be applied.

Figure 17-8 provides a direct solution for the correction factor, C, based on Equations 17.6 and 17.7.

The stability factor is used to reflect the level of uncertainty in the hydraulic conditions at a particular site. Equation 17.4 is based on the assumption of uniform or gradually varying flow. In many instances, this assumption is violated or other uncertainties come to bear; for example, debris and/or ice impacts or the cumulative effect of high shear stresses and forces from wind and/or boat generated waves. The stability factor is used to increase the design rock size when these conditions must be considered. Table 17-3 presents guidelines for the selection of an appropriate value for the stability factor.

17.7.1.2.3 Application

Application of the relationship in Equation 17.4 is limited to uniform or gradually varying flow conditions that are in straight or mildly curving channel reaches of relatively uniform cross section; however, use of the relationship may be necessary in non-uniform, rapidly varying flow conditions often present in natural channels with sharp bends and steep slopes and in the vicinity of bridge piers and abutments.

To fill the need for a design relationship that can be applied at sharp bends and on steep slopes in natural channels and at bridge abutments, it is recommended that Equation 17.4 be used with appropriate adjustments in velocity and/or stability factor as outlined in the following Sections.

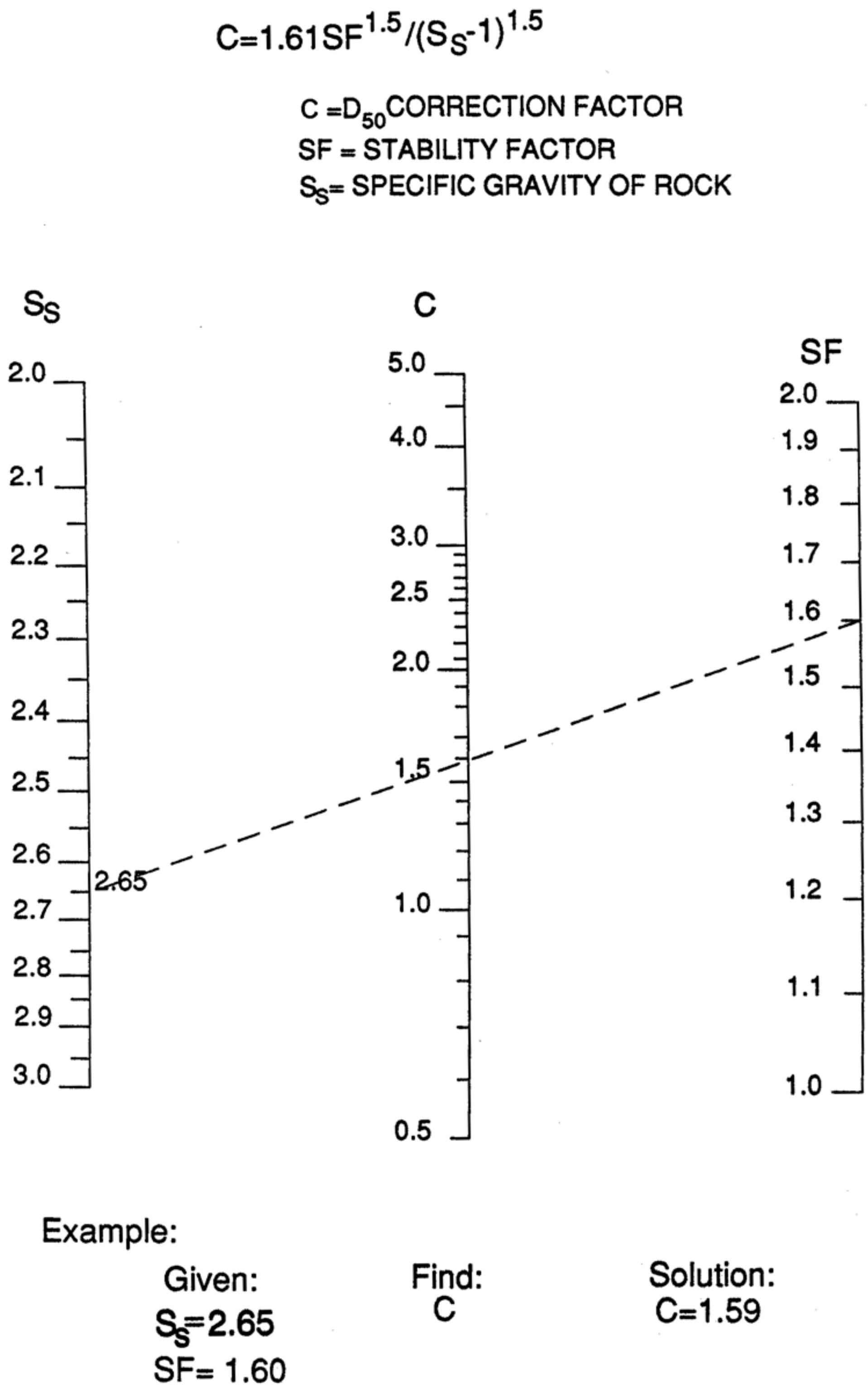


FIGURE 17-8 — Correction Factor For D_{50} Riprap Size

TABLE 17-3 — Guidelines for the Selection of Stability Factors

Condition	Stability Factor Range
Uniform flow; straight or mildly curving reach (curve radius/channel width > 30); impact from wave action and floating debris is minimal; little or no uncertainty in design parameters.	1.0 – 1.2
Gradually varying flow; moderate bend curvature (30 > curve radius/channel width > 10); impact from waves or floating debris moderate.	1.21 – 1.6
Approaching rapidly varying flow; sharp bend curvature (10 > curve radius/channel width); significant impact potential from floating debris and/or ice; significant wind and/or boat-generated waves (1 ft – 2 ft); high-flow turbulence; turbulently mixing flow at bridge abutments; significant uncertainty in design parameters.	1.61 – 2.0

17.7.1.2.4 Steep Slopes

Flow conditions in steep-sloped channels are rarely uniform and are characterized by high-flow velocities and significant flow turbulence. In applying Equation 17.4 to steep-sloped channels, care must be exercised in the determination of an appropriate velocity. When determining the flow velocity in steep-sloped channels, it is recommended that Equation 17.8 be used to determine the channel roughness coefficient. It is important to thoughtfully consider the guidelines for selection of stability factors as presented in Table 17-3.

On high-gradient streams, it is extremely difficult to obtain a good estimate of the median bed material size. For high-gradient streams with slopes greater than 0.002 and bed material larger than 0.2 ft (gravel, cobble or boulder size material), it is recommended that the relationship given in the following Equation be used to evaluate the base Manning's n:

$$n = 0.39 S_f^{0.38} R^{-0.16} \tag{17.8}$$

where: S_f = friction slope, ft/ft
R = hydraulic radius, ft

Jarrett *(9)* states the following limitations for the use of Equation 17.8:

1. The Equation is applicable to natural main channels having stable bed and bank materials (gravels, cobbles and boulders) without backwater.

2. The Equation can be used for slopes from 0.002 to 0.04 and for hydraulic radii from 0.5 ft to 7 ft. The upper limit on slope is due to a lack of verification data available for the slopes of high-gradient streams. Results of the regression analysis indicate that for hydraulic radii greater than 7 ft, n did not vary significantly with depth; thus, extrapolating to larger flows should not be too much in error, if the bed and bank material remain fairly stable.

3. Hydraulic radius does not include the wetted perimeter of bed particles.

4. Equation 17.8 is applicable to streams having relatively small amounts of suspended sediment.

17.7.1.2.5 Bridge Piers

For recommendations, see Appendix 10.B.

17.7.1.2.6 Wave Erosion

Waves generated by wind or boat traffic have been observed to cause bank erosion on inland waterways. The most widely used measure of riprap's resistance to waves is that developed by Hudson *(8)*. The so-called Hudson relationship is given by the following Equation:

$$W_{50} = (\gamma_s H^3) / (2.20 [S_s - 1]^3 \cot \theta) \tag{17.9}$$

where: W_{50} = weight of the median particle, lb

γ_s = unit weight of riprap (solid) material, lb/ft^3; the other parameters are as defined previously

H = the wave height, ft

S_s = specific gravity of riprap material

θ = bank angle with the horizontal

Assuming:

S_s = 2.65 and γ_s = 165 lb/ft^3, Equation 17.9 can be reduced to:

$$W_{50} = 16.7H^3/\cot\theta \tag{17.10}$$

In terms of an equivalent diameter, Equation 17.10 can be reduced to:

$$D_{50} = 0.75H/\cot^{1/3}\theta \tag{17.11}$$

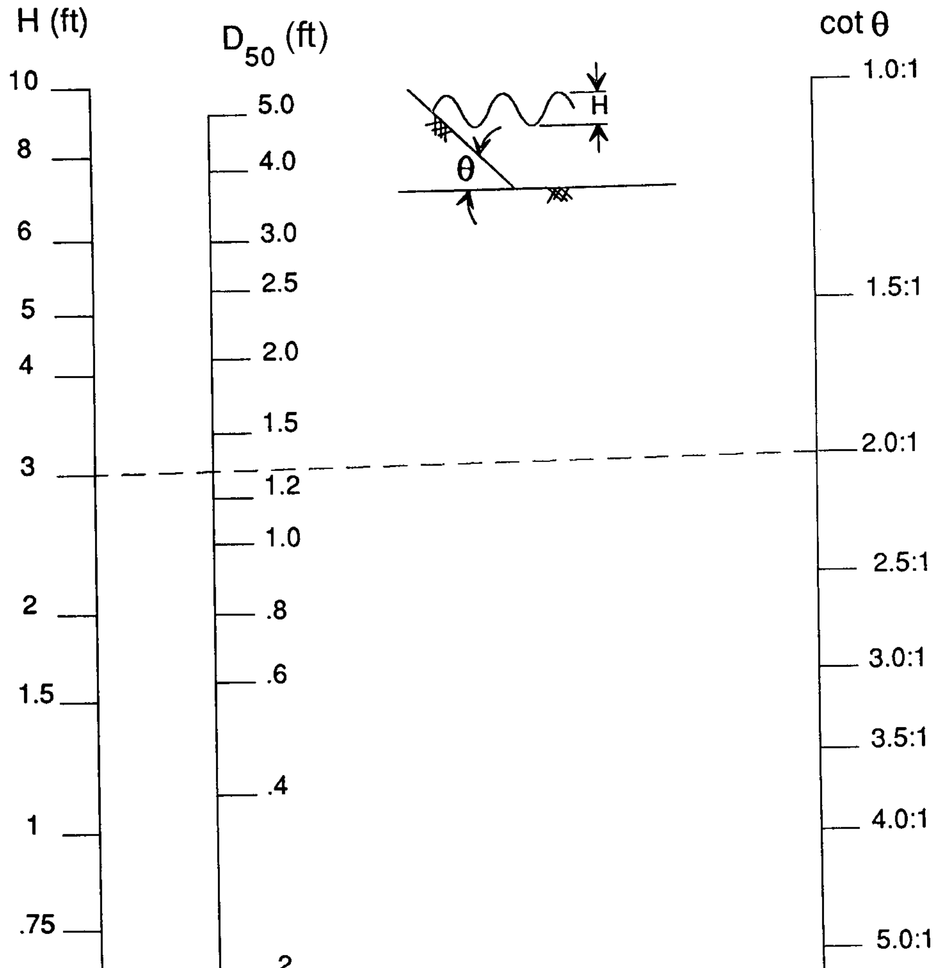
where: D_{50} = median riprap size, ft

Methods for estimating a design wave height are presented in Section 17.6.7. Equation 17.11 is presented in nomograph form in Figure 17-9. Equations 17.10 and 17.11 can be used for preliminary or final design when H is less than 5 ft, and there is no major overtopping of the embankment.

17.7.1.2.7 Ice Damage

Ice can affect riprap linings in a number of ways. Moving surface ice can cause crushing and bending forces and large impact loadings. The tangential flow of ice along a riprap-lined channel bank can also cause excessive shearing forces. Quantitative criteria for evaluating the impact ice has on channel protection schemes are unavailable. However, historic observations of ice flows in New England rivers indicate that riprap sized to resist design flow events will resist ice forces.

For design, consideration of ice forces should be evaluated on a case-by-case basis. In most instances, ice flows are not of sufficient magnitude to warrant detailed analysis. Where ice flows have caused problems, a stability factor of 1.2 to 1.5 should be used to increase the design rock size. Note that the selection of an appropriate stability factor to account for ice-generated erosive problems should be based on local experience.



$$D_{50} = 0.75 \frac{H}{\cos 1/3\ \theta}$$

D_{50} =Median Riprap Size

H = Wave height

θ = Bank Angle with Horizontal

Example

Given:	Find:	Solution:
$\cot\theta$ = 2:1 H = 3 ft.	D_{50}	D_{50}= 1.33 ft.

FIGURE 17-9 — Hudson Relationship for Riprap Size Required to Resist Wave Erosion

17.7.1.3 Rock Gradation

The gradation of stones in riprap revetment affects the riprap's resistance to erosion. The stone should be reasonably well graded throughout the riprap layer thickness. The gradation limits should not be so restrictive that production costs would be excessive. Table 17-4 presents suggested guidelines for establishing gradation limits. Table 17-5 presents six suggested gradation classes based on AASHTO specifications. Figure 17-10 can be used as an aid in selecting appropriate gradation limits.

It is recognized that the use of a four-point gradation as specified in Table 17-4 might be too harsh a specification for some smaller quarries. If this is the case, the 85% specification can be dropped as is done in Table 17-5. In most instances, a uniform gradation between D_{50} and D_{100} will result in an appropriate D_{85}.

17.7.1.4 Layer Thickness

All stones should be contained reasonably well within the riprap layer thickness to provide maximum resistance against erosion. Oversize stones, even in isolated spots, may cause riprap failure by precluding mutual support between individual stones, providing large voids that expose filter and bedding materials and creating excessive local turbulence that removes smaller stones. Small amounts of oversize stone should be removed individually and replaced with proper size stones. The following criteria apply to the riprap layer thickness:

1. It should not be less than the spherical diameter of the D_{100} (W_{100}) stone or less than 2.0 times the spherical diameter of the D_{50} (W_{50}) stone, whichever results in the greater thickness.

2. It should not be less than 12 in for practical placement.

3. The thickness determined by either No. 1 or No. 2 should be increased by 50% where the riprap is placed underwater to provide for uncertainties associated with this type of placement.

4. An increase in thickness of 6 in to 12 in, accompanied by an appropriate increase in stone sizes, should be provided where riprap revetment will be subject to attack by floating debris or ice or by waves from boat wakes, wind or bedforms.

17.7.1.5 Filter Design

A filter is a transitional layer of gravel, small stone or fabric placed between the underlying soil and the structure. The filter prevents the migration of the fine soil particles through voids in the structure, distributes the weight of the armor units to provide more uniform settlement and permits relief of hydrostatic pressures within the soils. A filter should be used whenever the riprap is placed on non-cohesive material subject to significant subsurface drainage (e.g., in areas where water surface levels fluctuate frequently and in areas of high groundwater levels).

17.7.1.5.1 Granular Filters

For rock riprap, a filter ratio of 5 or less between layers will usually result in a stable condition. The filter ratio is defined as the ratio of the 15% particle size (D_{15}) of the coarser layer to the 85% particle size (D_{85}) of the finer layer. An additional requirement for stability is that the ratio of the 15% particle size of the coarser material to the 15% particle size of the finer material should exceed 5 but be less than 40. These requirements can be stated as:

TABLE 17-4 — Rock Riprap Gradation Limits

Stone Size Range (ft)	Stone Weight Range (lb)	Percent of Gradation Smaller Than
$1.5D_{50}$ to $1.7D_{50}$	$3.0W_{50}$ to $5.0W_{50}$	100
$1.2D_{50}$ to $1.4D_{50}$	$2.0W_{50}$ to $2.75W_{50}$	85
$1.0D_{50}$ to $1.15D_{50}$	$1.0W_{50}$ to $1.5W_{50}$	50
$0.4D_{50}$ to $0.6D_{50}$	$0.1W_{50}$ to $0.2W_{50}$	15

TABLE 17-5 — Riprap Gradation Classes

Riprap Class	Rock Size[1] (ft)	Rock Weight[2] (lb)	Percent of Riprap Smaller Than
Facing	1.30 0.95 0.40	200 75 5	100 50 10
Light	1.80 1.30 0.40	500 200 5	100 50 10
1/4 ton	2.25 1.80 0.95	1000 500 75	100 50 10
1/2 ton	2.85 2.25 1.80	2000 1000 500	100 50 5
1 ton	3.60 2.85 2.25	4000 2000 1000	100 50 5
2 ton	4.50 3.60 2.85	8000 4000 2000	100 50 5

[1] *Assuming a specific gravity of 2.65.*

[2] *Based on AASHTO gradations.*

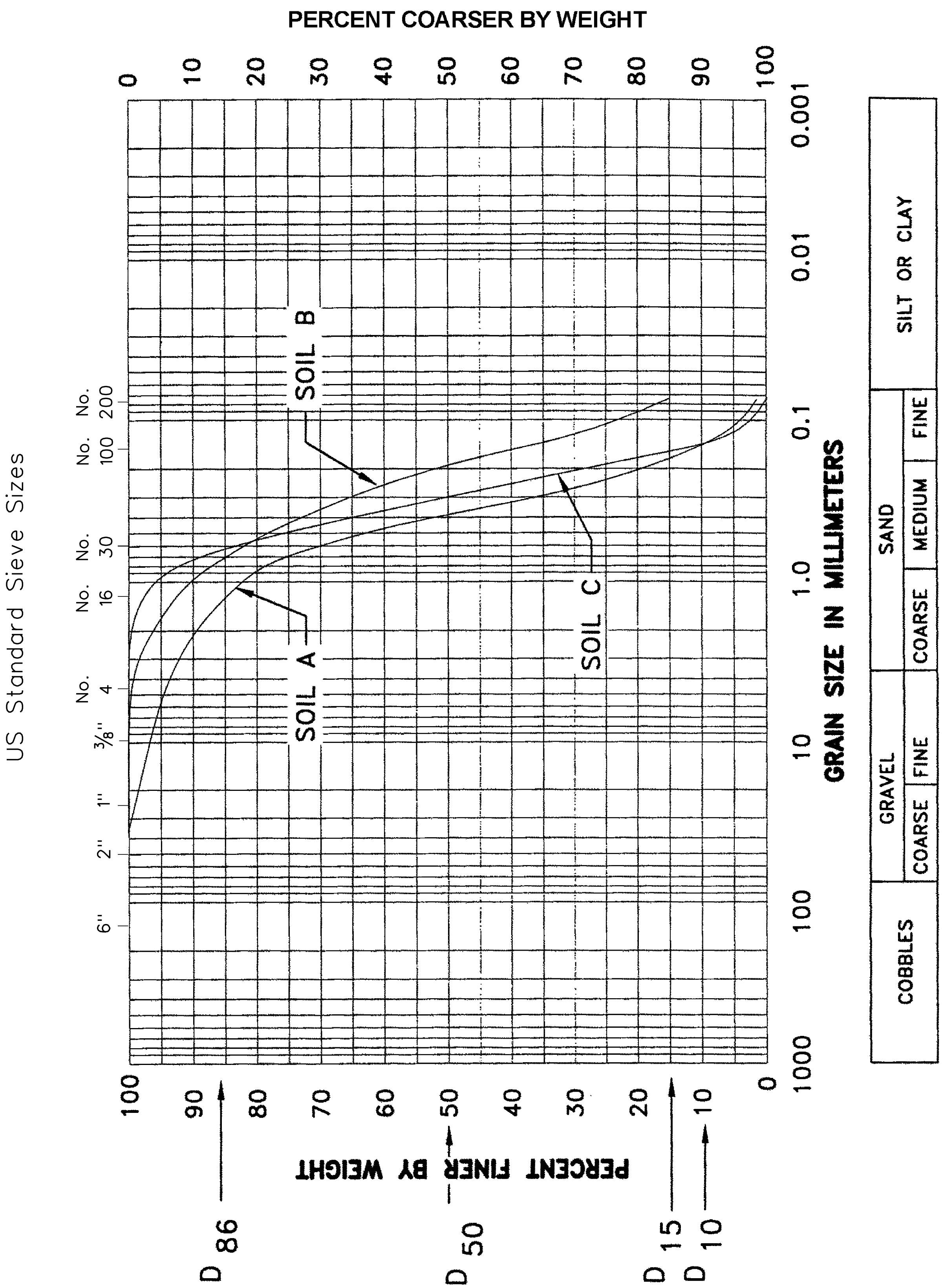


FIGURE 17-10 — Material Gradation

$$\frac{D_{15}\ (coarser\ layer)}{D_{85}\ (\ finer\ layer)} < 5 < \frac{D_{15}\ (coarser\ layer)}{D_{15}\ (\ finer\ layer)} < 40$$

The first test of the inequality is intended to prevent piping through the filter, the left portion of the second test provides for adequate permeability for structural bedding layers, and the right portion provides a uniformity criterion.

If a single layer of filter material will not satisfy the filter requirements, one or more additional layers of filter material must be used. The filter requirement applies between the bank material and the filter blanket, between successive layers of filter material if more than one layer is used, and between the filter blanket and the riprap cover. In addition to the filter requirements, the grain size curves for the various layers should be approximately parallel to minimize the infiltration of fine material from the finer layer to the coarser layer. Not more than 5% of the filter material should pass the No. 200 sieve. Figures 17-10 and 17-11 can be used as an aid in designing an appropriate granular filter.

The thickness of the filter blanket should range from 6 in to 18 in for a single layer or from 4 in to 8 in for individual layers of a multiple-layer blanket. Where the gradation curves of adjacent layers are approximately parallel, the thickness of the blanket layers should approach the minimum.

The thickness of individual layers should be increased above the minimum as the gradation curve of the material comprising the layer departs from a parallel pattern.

17.7.1.5.2 Geotextile Filters

Synthetic geotextile filters have found considerable use as alternatives to granular filters. Since the original geotextile erosion control application in 1957, thousands of successful projects have been completed. The following list of advantages relevant to using geotextile filters has been identified:

- Installation is generally quick and labor-efficient.
- Geotextile filters are more economical than granular filters.
- Geotextile filters have a more consistent and more reliable material quality.
- Geotextile filters have good inherent tensile strength.

Disadvantages include the following:

- Geotextiles can be difficult to install underwater.
- Geotextiles have widely variable hydraulic properties and must be designed based on project-specific conditions and performance requirements.
- Geotextile filter performance is sensitive to construction procedures.
- Special installation and inspection procedures may be necessary when using geotextile filters.

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DESCRIPTION ____________________ Checked by/Date: ___/___ ______

____________________ Sheet _____ of _____

GRANULAR FILTER:

LAYER	DESCRIPTION	D_{15} (ft)	D_{85} (ft)	RATIO	$\frac{D_{15}\text{COARSE}}{D_{85}\text{FINE}}$	< 5 <	$\frac{D_{15}\text{COARSE}}{D_{15}\text{FINE}}$	< 40

SUMMARY:

LAYER DESCRIPTION	D_{15}	D_{85}	THICKNESS

FABRIC FILTER: PHYSICAL PROPERTIES CLASS: ____________________

HYDRAULIC PROPERTIES

PIPING RESISTANCE < 50% PASSING #200 AOS < 0.6 mm

< 50% PASSING #200 AOS < 0.3 mm

PERMEABILITY SOIL PERMEABILITY < FABRIC PERMEABILITY

SELECTED FABRIC FILTER SPECIFICATIONS: ____________________

FIGURE 17-11 — Filter Design

17.7.1.5.3

Geotextile Filter Design

The design of geotextile filters closely follows traditional graded granular filter design principles and should consider the following performance areas:

- soil retention (piping resistance),
- permeability,
- clogging, and
- survivability.

It is extremely desirable that individual site conditions and performance requirements be established in conjunction with the geotextile design. Generalized geotextile requirements should be used only on very small or non-critical/non-severe installations where a detailed analysis is not warranted. AASHTO has developed materials and construction specifications (AASHTO Specification M-288) for routine, non-critical/non-severe geotextile applications. Details of geotextile filter design, for all levels of project severity and criticality, are presented in Reference *(4).* This reference provides detailed guidance on specifying and installing geotextiles for a variety of transportation applications. The American Society for Testing Material Committee D-35 has developed standard testing procedures for approximately 35 general, index and performance properties of geosynthetics. These standard test procedures are recommended for use in design and specifications when using geosynthetics.

The following design Steps are necessary for geotextile design in riprap and other permanent erosion control applications:

Step 1 Evaluate the application site (determine if the application is critical or severe):

- Critical applications involve high risk of loss of life, high potential for significant structural damage, or where repair costs would greatly exceed installation costs.
- Severe conditions include draining gap-graded, pipable or dispersible soil, high hydraulic gradients or dynamic, cyclic or pulsating flow conditions.

Step 2 Obtain and test soil samples (perform grain size analysis and permeability tests).

Step 3 Evaluate possible armor material and placement procedures.

Step 4 Calculate anticipated reverse flow through the erosion control system.

Step 5 Determine geotextile requirements:

a. soil retention,
b. permeability/permittivity,
c. clogging, and
d. survivability.

Step 6 Estimate cost and prepare specifications.

17.7.1.5.4 Geotextile Installation Procedures

To provide good performance, a properly selected cloth should be installed with due regard for the following precautions:

- Grade area and remove debris to provide a smooth, fairly even surface.
- Place geotextile loosely, laid with the machine (generally roll) direction in the direction of anticipated water flow or movement.
- Seam the geotextile or use a minimum overlap of 1 ft.
- The maximum allowable slope on which a riprap-geotextile system can be placed is equal to the lowest soil-geotextile friction angle for the natural ground or stone-geotextile friction angle for cover (armor) materials. Additional reductions in slope may be necessary due to hydraulic considerations and possible long-term stability. For slopes greater than 1V:2.5H, special construction procedures will be required.
- For streambank and wave action applications, the geotextile must be keyed in at the bottom of the slope. See Figure 17-12. If the system cannot be extended a few feet above the anticipated high-water level, the geotextile should also be keyed in at the crest of the slope.
- Place the revetment (cushion layer and/or riprap) over the geotextile width, while avoiding puncturing it.

17.7.1.6 Edge Treatment

The edges of riprap revetments (flanks, toe and head) require special treatment to prevent undermining. The flanks of the revetment should be designed as illustrated in Figure 17-13. The upstream flank is illustrated in Section A-A and the downstream flank in Section B-B of this Figure. A more constructible flank section uses riprap rather than compacted fill.

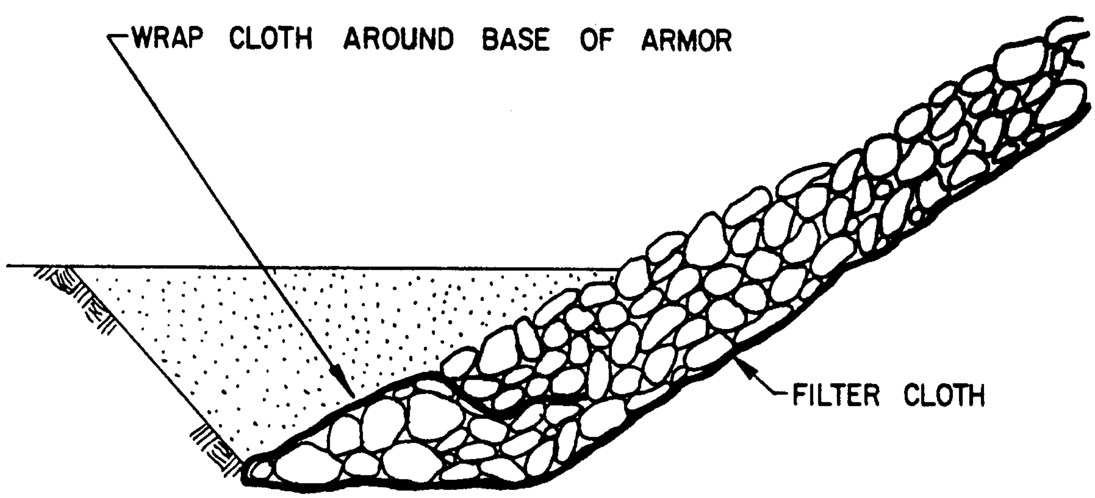

FIGURE 17-12 — Geotextile Filters

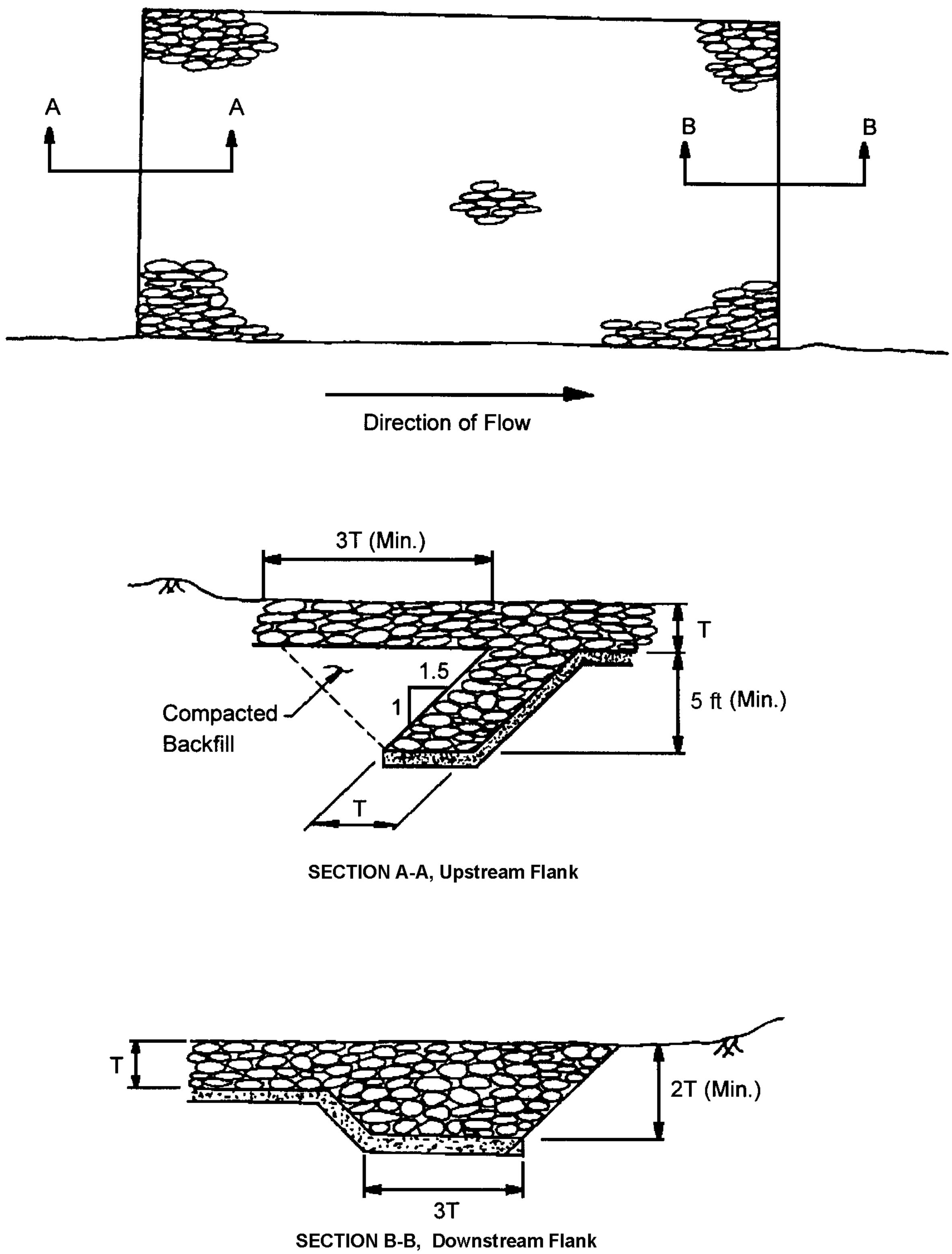

FIGURE 17-13 — Typical Riprap Installation: Plan and Flank Details

Undermining of the revetment toe is one of the primary mechanisms of riprap failure. The toe of the riprap should be designed as illustrated in Figure 17-14. The toe material should be placed in a toe trench along the entire length of the riprap blanket.

Where a toe trench cannot be dug, the riprap blanket should terminate in a thick, stone toe at the level of the streambed (see alternative design in Figure 17-14). Care must be taken during the placement of the stone to ensure that the toe material does not mound and form a low dike; a low dike along the toe could result in flow concentration along the revetment face that could stress the revetment to failure. In addition, care must be exercised to ensure that the channel's design capability is not impaired by too much riprap in a toe mound.

The size of the toe trench or the alternative stone toe is controlled by the anticipated depth of scour along the revetment. As scour occurs (and in most cases it will), the stone in the toe will launch into the eroded area as illustrated in Figure 17-15. Observation of the performance of these types of rock toe designs indicates that the riprap will launch to a final slope of approximately 1V:2H.

The volume of rock required for the toe must be equal to or exceed 1½ times the volume of rock required to extend the riprap blanket (at its design thickness and on a slope of 1V:2H) to the anticipated depth of scour. Dimensions should be based on the required volume using the thickness and depth determined by the scour evaluation. The alternative location can be used where the amount of rock required would not constrain the channel. Establishing a design scour depth is covered in Section 17.6.7.

17.7.1.7 Construction Considerations

The construction considerations related to the construction of riprap revetments include bank slope or angle, bank preparation and riprap placement.

17.7.1.7.1 Bank Preparation

The bank should be prepared by first clearing all trees and debris from the bank and grading the bank surface to the desired slope. In general, the graded surface should not deviate from the specified slope line by more than 6 in. However, local depressions larger than this can be accommodated because initial placement of filter material and/or rock for the revetment will fill these depressions. In addition, any large boulders or debris found buried near the edges of the revetment should be removed.

17.7.1.7.2 Riprap Placement

The common methods of riprap placement are hand placing; machine placing, such as from a skip, dragline or some form of bucket; and dumping from trucks and spreading by bulldozer. Hand placement produces the best riprap revetment, but it is the most expensive method except when labor is unusually cheap. Steeper side slopes can be used with hand-placed riprap than with other placing methods. Where steep slopes are unavoidable (where channel widths are constricted by existing bridge openings or other structures, and where rights-of-way are costly), hand placement should be considered.

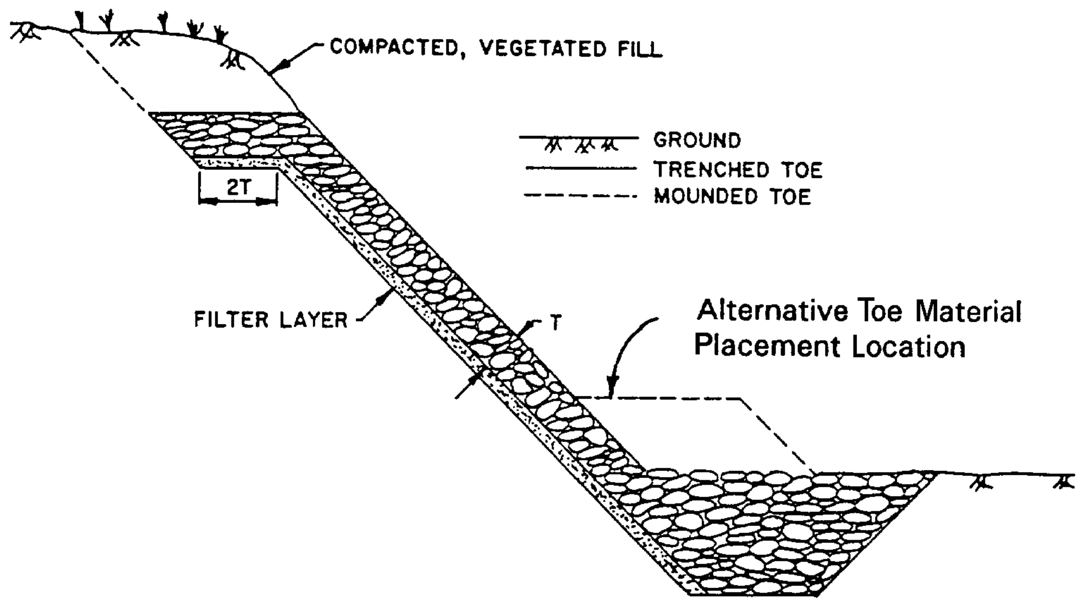

FIGURE 17-14 — Typical Riprap Installation: End View (Bank Protection Only)

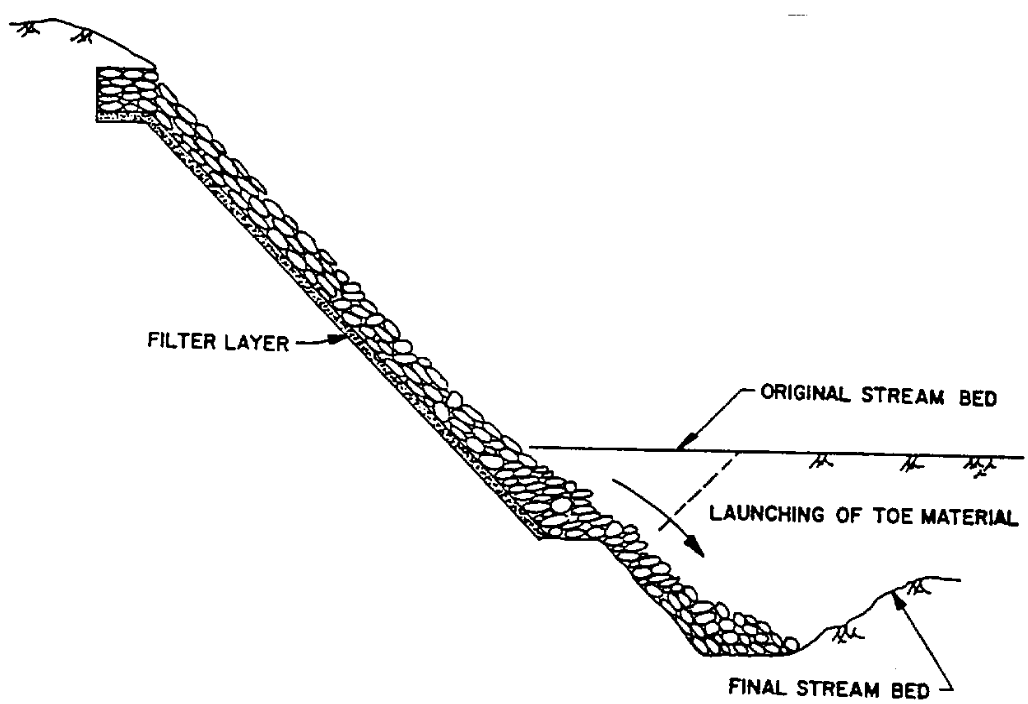

FIGURE 17-15 — Launching of Riprap Toe Material

In the machine-placement method, sufficiently small increments of stone should be released as close to their final positions as practical. Rehandling or dragging operations to smooth the revetment surface tend to result in segregation and breakage of stone and can result in a rough revetment surface. Stone should not be dropped from an excessive height since this may result in the same undesirable conditions. Riprap placement by dumping with spreading is satisfactory provided that the required layer thickness is achieved. Riprap placement by dumping and spreading is not recommended because a large amount of segregation and breakage can occur. In some cases, it may be economical to increase the layer thickness and stone size somewhat to offset the shortcomings of this placement method.

17.7.1.8 Design Procedure

Rock riprap design procedures outlined in the following Sections are comprised of three primary Sections — preliminary data analysis, rock sizing and revetment detail design. The individual Steps in the procedure are numbered consecutively throughout each of the Sections. Figures 17-16 and 17-17 provide a useful format for recording data at each Step of the analysis. *Note: The rock-sizing procedures described in the following Steps are designed to prevent riprap failure from particle erosion.*

17.7.1.8.1 Preliminary Data

Step 1 Compile all necessary field data including channel cross section surveys, soils data, aerial photographs and history of problems at site.

Step 2 Determine design discharge.

Step 3 Develop design cross section(s).

Step 4 Compute design water surface:

a. When evaluating the design water surface, Manning's n should be estimated. If a riprap lining is being designed for the entire channel perimeter, an estimate of the rock size may be required to determine the roughness coefficient n (Section 17.6.6).

b. If the design section is a regular trapezoidal shape and flow can be assumed to be uniform, use design procedures from the Channels Chapter.

c. If the design section is irregular or flow is not uniform, backwater procedures must be used to determine the design water surface.

d. Any backwater analysis conducted must be based on conveyance weighting of flows in the main channel, right bank and left bank.

Step 5 Determine design average velocity and depth:

a. Average velocity and depth should be determined for the design section in conjunction with the computations of Step 4. In general, the average depth and velocity in the main flow channel should be used.

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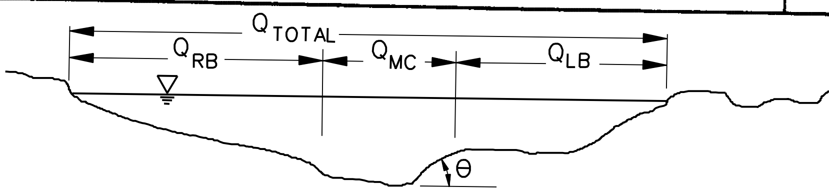
Definition Sketch:



Q_{TOTAL} ______
Q_{MC} ______
Q_{LB} ______
Q_{RB} ______

Soil Characteristics:
D_{15} ______
D_{50} ______
D_{85} ______

DEPTH OR W.S. (ft) ①	A (ft²) ②	V_a (ft/sec) ③	d_a (ft) ④	Θ ⑤	Φ ⑥	K_1 ⑦	D_{50} (ft) ⑧	SF ⑨	S_s ⑩	C ⑪	$C_{P/A}$ ⑫	D_{50} (ft) ⑬	NOTES ⑭

Design Sketch

RIPRAP CHARACTERISTICS:

Size:
D_{50} ______
Class ______

Thickness:
$2D_{50}$ ______
D_{100} ______
Use ______

AASHTO Gradation:

Size: (ft)	Percent Finer
______	100
______	50
______	5-10

FABRIC CHARACTERISTICS:

Granular:

Size (ft)	Percent Finer
______	85
______	50
______	15

Fabric:
AOS* < ______
Perm.> ______

*Average Opening Size

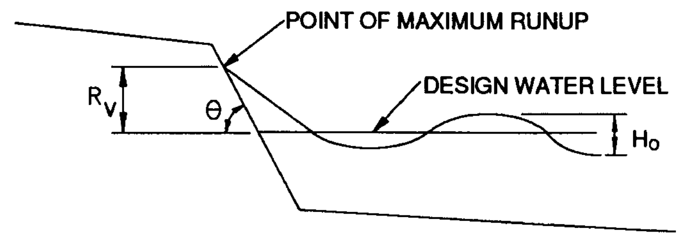
① Water Surface Elevation
② Main channel flow area
③ Main channel average velocity
④ Main channel average depth
⑤ Bank angle
⑥ Riprap angle of repose (Chart 4)
⑦ Bank angle correction (Chart 3)
⑧ Riprap size (Chart 1)
⑨ Stability Factor
⑩ Riprap specific gravity
⑪ Riprap size correction factor (Chart 2)
⑫ Pier/Abutment Correction (3.38 if applicable)
⑬ Correction D_{50} = ⑧ · ⑪ · ⑫
⑭ Notes or Comments

FIGURE 17-16 — Riprap Data Sheet for Channels

PROJECT ______	Prepared by/Date: : ______/______
DESCRIPTION ______	Checked by/Date: ______/______
______	Sheet ____ of ____

WIND SPEED (mph)	FETCH (mi)	H_0 (ft) ①	θ	$\frac{R_V}{H_o}$ ②	CORR. FACTOR ③	R_V (ft) ④	D_{50} (ft) ⑤

DEFINITION SKETCH



RIPRAP SIZE:

D_{50} ______ ft
CLASS ______

REVETMENT THICKNESS:

$2D_{50}$ ______ ft
D_{100} ______ ft
USE ______ ft

AASHTO GRADATION:

SIZE (ft)	PERCENT FINER
______	100
______	50
______	5-10

① USE FIGURE 17-4 OR OTHER METHOD
② USE FIGURE 17-3
③ SEE TABLE 17-1
④ ① x ② x ③
⑤ USE FIGURE 17-9

FIGURE 17-17 — Riprap Data Sheet for Wave Action

b. If riprap is being designed to protect channel banks, abutments or piers located in the floodplain, then average floodplain depths and velocities should be used.

Step 6 Compute the bank angle correction factor, K_1 (Equation 17.5, Figures 17-5 and 17-6).

Step 7 Determine riprap size required to resist particle erosion (Equation 17.4, Figure 17-4):

a. Initially assume no corrections.

b. Evaluate correction factor for rock-riprap specific gravity and stability factor ($C = C_{sg}C_{sf}$).

c. If designing riprap for piers or abutments, see Bridge Chapter.

Step 8 If entire channel perimeter is being stabilized and an assumed D_{50} was used in determination of Manning's n for backwater computations, repeat Steps 4 through 7.

Step 9 If surface waves are to be evaluated:

a. Determine significant wave height (see Coastal Zone Chapter).

b. Use Figure 17-9 to determine rock size required to resist wave action (Equation 17.11).

Step 10 Select final D_{50} riprap size, set material gradation (see Section 17.7.1.3 and Figure 17-10) and determine riprap layer thickness (see Section 17.7.1.4). If final D_{50} riprap size is different than that derived from Step 8, repeat Steps 4 through 10.

Step 11 Determine longitudinal extent of protection required (Section 17.6.7).

Step 12 Determine appropriate vertical extent of revetment (Section 17.6.7).

Step 13 Design filter layer (Section 17.7.1.5, Figure 17-11):

a. Determine appropriate filter material size and gradation.
b. Determine layer thickness.

Step 14 Design edge details (flanks and toe) (Section 17.7.1.6).

17.7.1.9 Design Examples

The following design Examples illustrate the use of the design methods and procedures outlined above. Two Examples are given. Example 1 illustrates the design of a riprap-lined channel section. Example 2 illustrates the design of riprap as bank protection. In the Examples, the Steps correlate with the design procedure outline presented above. Computations are also shown on appropriate figures.

Example 1

A 1,250-ft channel reach is to be realigned to make room for the widening of an existing highway. Realignment of the channel reach will necessitate modifying the channel and reducing its length from 1,250 ft to 1,000 ft. The channel is to be sized to carry 5,000 ft^3/s within its banks. Additional site conditions are as follows:

- flow conditions can be assumed to be uniform or gradually varying;
- the existing channel profile dictates that the modified reach be designed at a uniform slope of 0.0049 ft/ft;
- the natural soils are gap graded from medium sands to coarse gravels giving the following distribution:

  D_{85} = 0.105 D_{50} = 0.064 ft D_{15} = 0.0045 ft
  k (permeability) = 3.5 x 10^{-4} m/s

- Available rock riprap has a specific gravity of 2.65 and D_{50} = 1.0 ft.

Design a stable, trapezoidal, riprap-lined channel for this site. Design figures used to summarize data in this Example are reproduced in Figures 17-18 and 17-19:

Step 1 Compile Field Data:

a. See given information for this Example.

b. Other field data would include site history, geometric constraints, roadway crossing profiles and site topography.

Step 2 Design Discharge:

a. Given as 5,000 ft^3/s.

b. Discharge in main channel equals the design discharge because entire design discharge is to be contained in channel as specified.

Step 3 Design Cross Section:

a. As specified, a trapezoidal section is to be designed.

b. Initially assume a trapezoidal section with 20-ft bottom width and 1V:2H side slopes (see Figure 17-18).

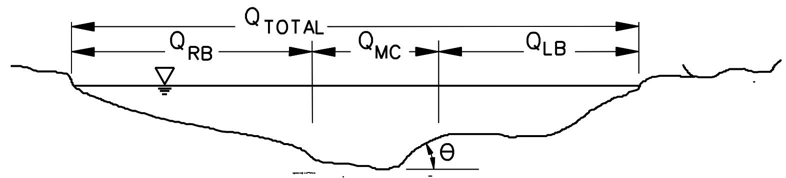
Step 4 Compute Design Water Surface:

Assume R = 7 ft:

Project Example 1
Description

Prepared by/date SAB / 2/4
Checked by/date CJH / 3/4
sheet of

Definition Sketch:

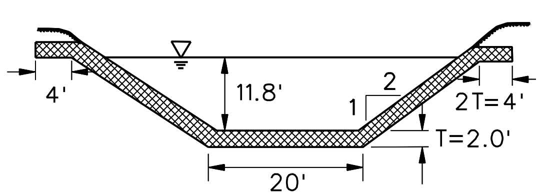


Q_{TOTAL} 5000
Q_{MC} 5000
Q_{LB}
Q_{RB}

Soil Characteristics:
D_{15} 0.0045 ft
D_{50} 0.04 ft
D_{85} 0.105 ft

DEPTH OR W.S. (ft) ①	A (ft^2) ②	V_a (ft/sec) ③	d_a (ft) ④	Θ ⑤	Φ ⑥	K_1 ⑦	D_{50} (ft) ⑧	SF ⑨	S_s ⑩	C ⑪	$C_{P/A}$ ⑫	D_{50} (ft) ⑬	NOTES ⑭
11.8	514.5	9.7	11.8	2:1	41°	0.73	0.43	1.2	2.65	1	N/A	0.43	Bank
							0.28	1.2	2.65	1	N/A	0.28	Bed

Design Sketch



RIPRAP CHARACTERISTICS:

Size:
D_{50} 0.95
Class Facing

Thickness:
$2D_{50}$ 1.9 ft
D_{100} 1.3 ft
Use 2.0 ft

AASHTO Gradation:

Size: (ft)	Percent Finer
1.30	100
0.95	50
0.40	5-10

FABRIC CHARACTERISTICS:

Granular:

Size (ft)	Percent Finer
0.20	85
0.17	50
0.10	15

Fabric:
AOS * <
Perm. >

* Average Opening Size

① Water Surface Elevation
② Main channel flow area
③ Main channel average velocity
④ Main channel average depth
⑤ Bank angle
⑥ Riprap angle of repose (Chart 4)
⑦ Bank angle correction (Chart 3)
⑧ Riprap size (Chart 1)
⑨ Stability Factor
⑩ Riprap specific gravity
⑪ Riprap size correction factor (Chart 2)
⑫ Pier/Abutment Correction (3.38 if applicable)
⑬ Correction D_{50} = ⑧ · ⑪ · ⑫
⑭ Notes or Comments

FIGURE 17-18 — Riprap Size Form (Example 1)

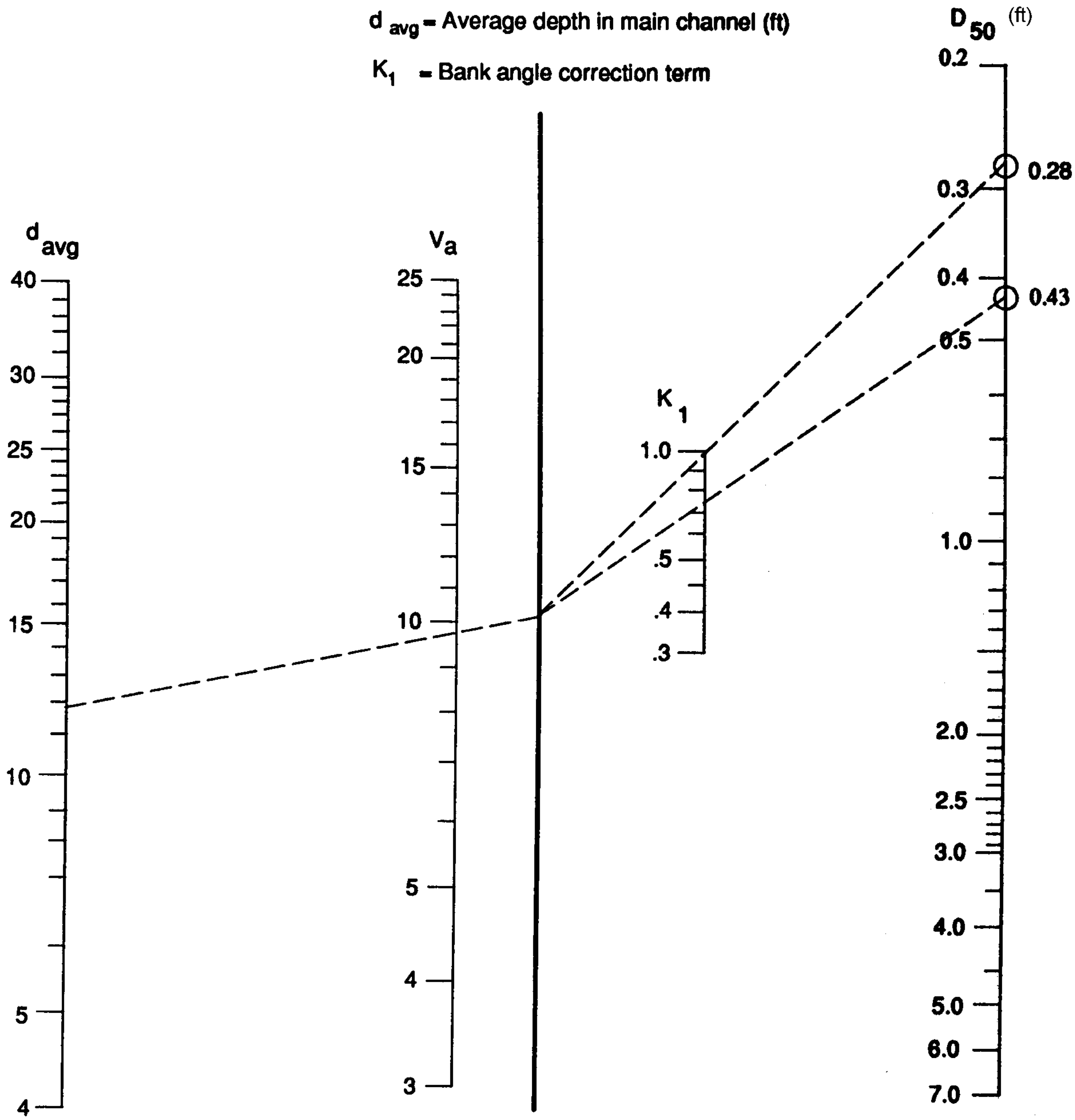
$$D_{50} = 0.001\, V_a^3 / (d_{avg}^{1/2} K_1^{3/2})$$

D_{50} = Median Riprap Size (ft.)

V_a = Average velocity in main channel (ft/sec)

d_{avg} = Average depth in main channel (ft)

K_1 = Bank angle correction term



Example

Given:

V_a = 9.7 (ft/s)

d_{avg} = 11.8 ft.

K_1 = 0.73

Find:

D_{50}

Solution:

D_{50} = 0.43 (ft)

FIGURE 17-19 — Riprap Size Relationship (Example 1, Step 7)

- Compute roughness coefficient using Equation 17.8:

  $n = 0.39S_f^{0.38}\ R^{-0.16}$
  $n = 0.39(0.0049)^{0.38}\ (7)^{*-0.16}$
  $n = 0.046$ (Use $n = 0.04$ for this Example)

  *Assume R = 7 ft

- Solve Manning's equation for normal depth or see Channels Chapter:

  $Q = (1.486/n)AR^{2/3}S^{1/2}$
  $d = 11.8$ ft

- Compute hydraulic radius to compare with the assumed value used in Step 4(a) (use computer programs, available charts and tables or manually compute):

  $R = A/P$
  $R = 514.5/72.8$
  $R = 7.1$ ft

  R = 7.1 ft which is approximately equal to R (assumed); therefore, d = 11.8 ft OK

Step 5 Determine Design Parameters:

$A = 11.8(20) + 2(11.8)^2 = 514.5\ ft^2$
$V_a = Q/A = 5{,}000/514.5 = 9.7$ ft/s
$d_a = d = 11.8$ ft (uniform channel bottom)

Step 6 Bank Angle Correction Factor:

θ = 1V:2H
$\phi = 41^o$ (from Figure 17-6)
$K_1 = 0.73$ (from Figure 17-5)

Step 7 Determine riprap size (see Section 17.7.1.2):

a. Using Figure 17-19:

   For channel bed: $D_{50} = 0.28$ ft
   For channel bank: $D_{50} = 0.43$ ft

b. Riprap specific gravity = 2.65 (given):

   Stability factor = 1.2 (Column 9, Figure 17-16)
   (uniform flow, little or no uncertainty in design)
   C = 1

c. No piers or abutments to evaluate for this Example; therefore:

   $C_{p/a} = 1$

d. Corrected riprap size:

For channel bed: $D'_{50} = D_{50} = 0.28$ ft
For channel banks: $D'_{50} = D_{50} = 0.43$ ft

Step 8 Not applicable.

Step 9 Surface waves: Surface waves determined not to be a problem at this site.

Step 10 Select Design Riprap Size, Gradation and Layer Thickness:

D_{50} size: Recommend AASHTO Face Class riprap
$D_{50} = 0.95$ ft (for entire perimeter)

Layer thickness (T):
$T = 2D_{50} = 2(0.95\text{ ft}) = 1.9$ ft, or $T = D_{100} = 1.3$ ft
Use T = 2.0 ft

Step 11 Longitudinal Extent of Protection:

Riprap lining to extend along entire length of modified reach plus one channel width upstream and 1.5 channel width downstream.

Step 12 Vertical Extent of Protection: Riprap entire channel perimeter to top-of-bank.

Step 13 Filter Layer Design:

a. Filter material size:

$$\frac{D_{15}\text{ [coarser layer]}}{D_{85}\text{ [finer layer]}} < 5 < \frac{D_{15}\text{ [coarser layer]}}{D_{15}\text{ [finer layer]}} < 40$$

For the riprap to soil interface:

$$\frac{D_{15}\text{ [riprap]}}{D_{85}\text{ [soil]}} = \frac{0.6}{0.100} = 6 > 5$$

and:

$$\frac{D_{15}\text{ [riprap]}}{D_{15}\text{ [soil]}} = \frac{0.6}{0.0045} = 133 > 40$$

Therefore, a filter layer is needed. Try 2-in uniformly graded coarse gravel filter. For the filter to soil interface:

$$\frac{D_{15}\text{ [filter]}}{D_{85}\text{ [soil]}} = \frac{0.100}{0.105} = 0.95 < 5$$

and:

$$\frac{D_{15}\ [\text{filter}]}{D_{15}\ [\text{soil}]} = \frac{0.100}{0.0045} = 22.2 > 5 \text{ and } < 40$$

Therefore, filter to soil interface is OK. For the riprap to filter interface:

$$\frac{D_{15}\ [\text{riprap}]}{D_{85}\ [\text{filter}]} = \frac{0.6}{0.200} = 3 < 5$$

and:

$$\frac{D_{15}\ [\text{riprap}]}{D_{15}\ [\text{filter}]} = \frac{0.6}{0.10} = 6 > 5 \text{ and } < 40$$

Therefore, the 2-in filter material is adequate.

b. Filter layer thickness:

Because soil gradation curve and filter layer gradation curve are not approximately parallel, use layer thickness of 8 in.

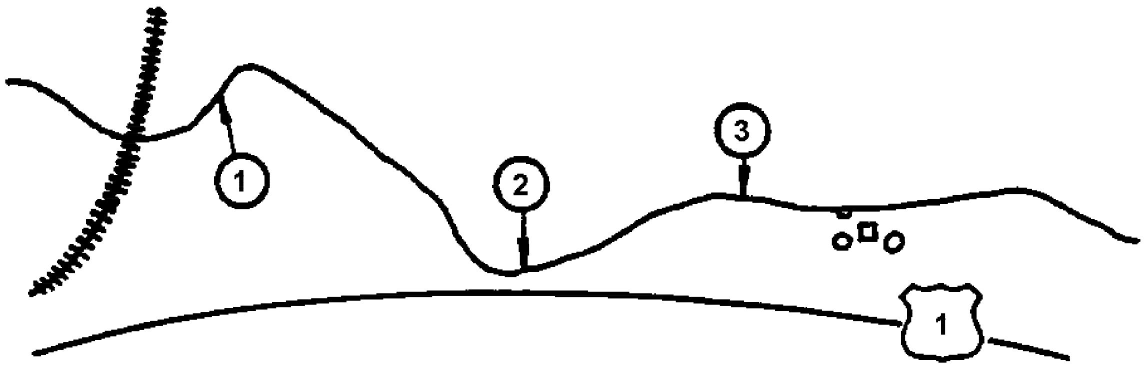
Step 14 Edge Details

Line entire perimeter; edge details as per Figure 17-13 (also see sketch on Figure 17-18).

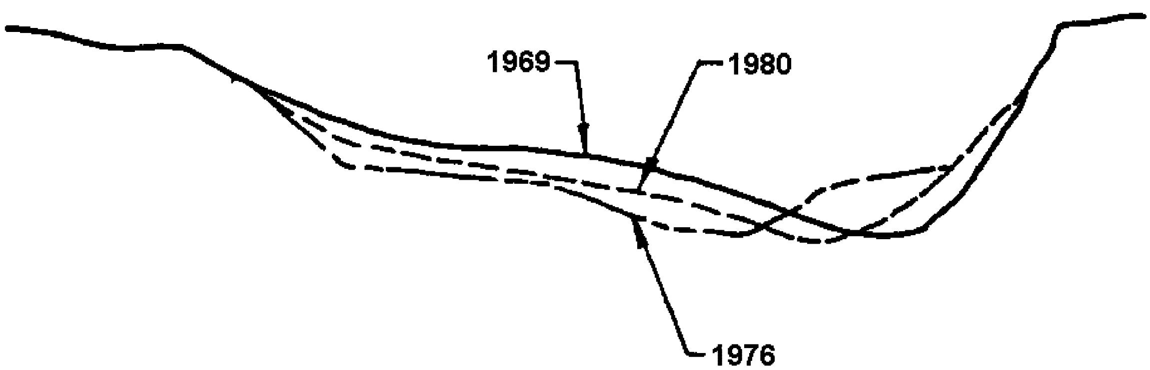
Example 2

The site illustrated in Figure 17-20 and discussed below is migrating laterally towards Route 1 (see Figure 17-20(a)). Design a riprap revetment to stabilize the active bank erosion at this site.

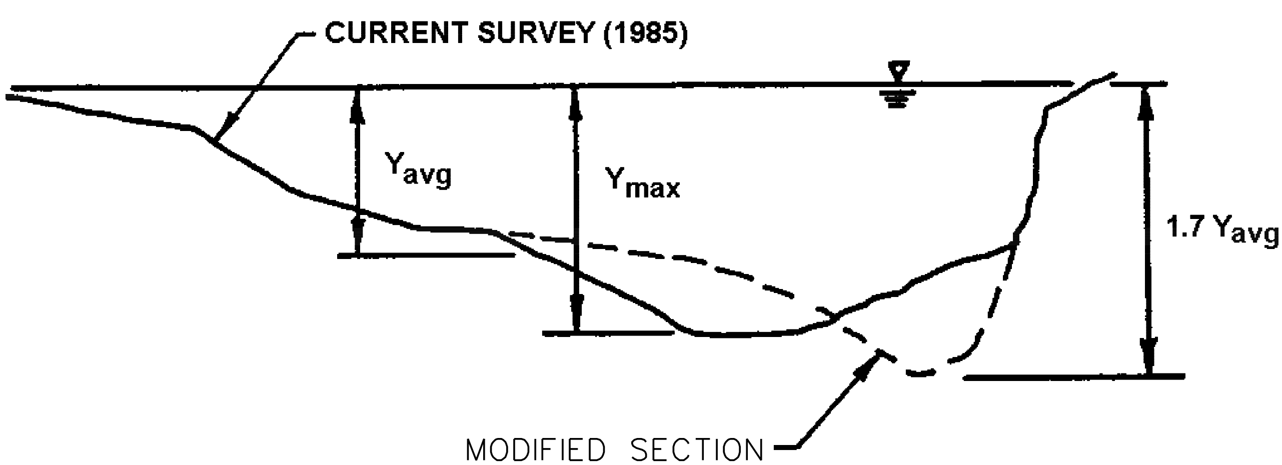
The process of developing an appropriate channel geometry is illustrated in Figure 17-20. Figure 17-20a illustrates the location of the design site at Position "2" along Route 1. The section illustrated in Figure 17-20c was surveyed at this location, and represents the current condition. No previous channel surveys were available at this site; however, data from several old surveys were available in the vicinity of a railroad crossing upstream (Location 1). Figure 17-20b illustrates these survey data. The surveys indicate that there is a trend for the thalweg of the channel to migrate within the right half of the channel. Because Locations 1 and 2 are along bends of similar radii, it can be reasonably assumed that a similar phenomenon occurs at Location 2. A thalweg located immediately adjacent to the channel bank reasonably represents the worst case hydraulically for the section at Location 2; therefore, the surveyed section at Location 2 is modified to reflect this. In addition, the maximum section depth (located in the thalweg) is increased to reflect the effect of stabilizing the bank. The maximum depth in the thalweg is set to 1.7 times the average depth of the original section (note that it is assumed that the average depth before modification of the section is the same as the average depth after modification). The final modified section geometry is illustrated in Figure 17-20c. Although Equation 17.3 is not applicable to channels with sharp bends, it is used in this Example to illustrate the design procedure.



(a) Site Location



(b) Cross Section at Location 1



(c) Cross Section and Modified Section at Location 2

FIGURE 17-20 — Channel Geometry Development (Example 2)

Additional site conditions are as follows:

- Flow conditions are gradually varying.
- Channel characteristics are as described above.
- Topographic survey indicates:
  - channel slope = 0.0024 ft/ft
  - channel width = 300 ft
- Bend radius = 1,200 ft.
- Channel bottom is armored with cobble-size material having a D_{50} of approximately 0.5 ft.
- Bank soils are silty sand with the following soil characteristics:

  D_{85} = 0.0042 ft
  D_{50} = 0.0015 ft
  D_{15} = 0.00045 ft
  k (permeability) = 1.0 x 10^{-6} m/s

- Available rock riprap has a specific gravity of 2.65 and is described as angular.
- Field observations indicate that the banks are severely cut just downstream of the bend apex; erosion was observed downstream to the bend exit and upstream to the bend quarter points.
- Bank height along cut banks is approximately 9 ft.

Design Figures used to summarize data in this Example are reproduced in Figures 17-21 and 17-22.

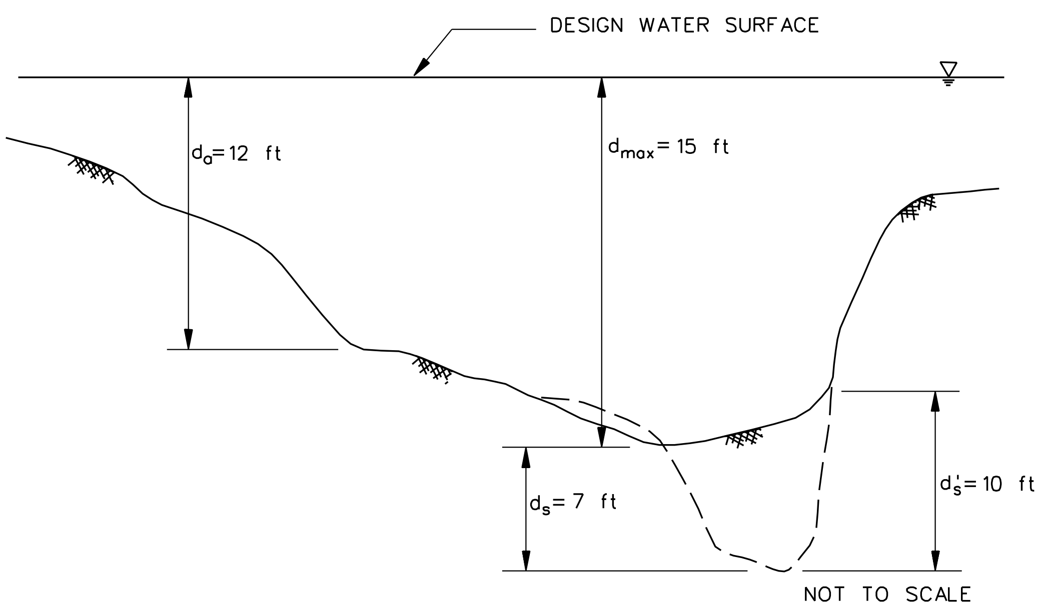


FIGURE 17-21 — Channel Cross Section For Example 2
Illustrating Flow and Potential Scour Depths

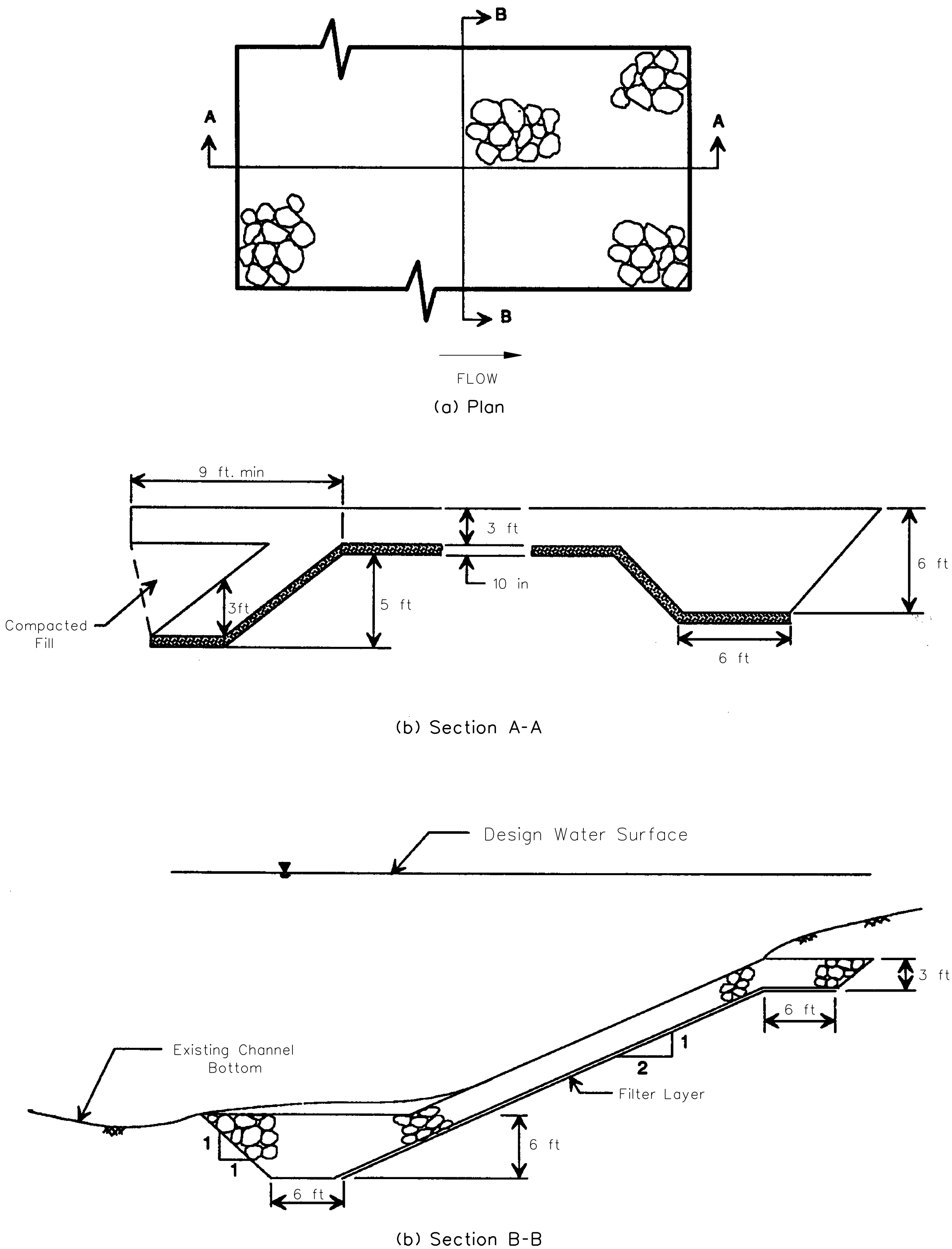


FIGURE 17-22 — Toe and Flank Details (Example 2)

Step 1 Compile Field Data:

- See given information for this Example.
- See site history given above.

Step 2 Design Discharge:

- Given as 46,700 ft^3/s.
- From backwater analysis of this reach, it is determined that the discharge confined to the main channel (Q_{mc}) is 34,700 ft^3/s.

Step 3 Design Cross Section:

- Only the channel bank is to be stabilized; therefore, the channel section will consist of the existing channel with the bank graded to an appropriate angle to support the riprap revetment. Figure 17-21 illustrates the existing channel section.
- To minimize loss of bank vegetation, and limit the encroachment of the channel on adjacent lands, a 1V:2H bank slope is to be used.
- As given, the current bank height along the cut banks is 9 ft.

Step 4 Compute Design Water Surface:

(a) Determine roughness coefficient using Equation 17.8. For this Example, n = 0.042. This represents the average reach “n” used in the backwater analysis.

(b) Compute flow depth:

- Flow depth determined from backwater analysis. The maximum main channel depth was determined to be d_{max} = 15.0 ft.
- Hydraulic radius for main channel:

  R = 10.4 ft (from backwater analysis)
  R assumed (10 ft) is approximately equal to R actual; therefore, “n” as computed is OK.

Step 5 Determine Other Design Parameters:

From backwater analysis (all main channel values):

$A = 2,750\ ft^2$
$V_a = 12.6$ ft/s
$d_a = d = 12.0$ ft/s

Step 6 Bank Angle Correction Factor:

θ = 1V:2H
ϕ = 41° (from Figure 17-6, Figure 17-22)
K_1 = 0.73 (from Figure 17-5)

Step 7 Determine Riprap Size:

(a) Using Figure 17-4: D_{50} = 0.9 ft

(b) Riprap specific gravity = 2.65 (given):

Stability factor = 1.6 (from Table 17-3)
(gradually varying flow, sharp bend — bend radius to width = 4)
C = 1.6

(c) No piers or abutments to evaluate for this Example; therefore:

$C_{p/a}$ = 1

(d) Corrected riprap size:

$D'_{50} = D_{50}(1.6)(0.9) = 1.44$ ft

Step 8 Not applicable

Step 9 Surface waves: Surface waves determined not to be a problem at this site.

Step 10 Select Design Riprap Size, Gradation and Layer Thickness (Preliminary Design of Waterway Area):

D_{50} size: Recommend 0.25 ton class riprap
D_{50} = 1.8 ft
Gradation: See Figure 17-23.
Layer thickness (T):
$T = 2D_{50} = 2(1.8) = 3.6$ ft, or $T = D_{100} = 2.25$ ft. Use T = 3.6 ft

Step 11 Longitudinal Extent of Protection:

Field observations indicate that the banks are severely cut just downstream of the bend apex; erosion was also observed downstream to the bend exit and upstream to the bend quarter points. Therefore, establish longitudinal limits of protection to extend to a point 300 ft (W) upstream of the bend entrance and to a point 450 ft (1.5 W) downstream of the bend exit.

Step 12 Vertical Extent of Protection:

Riprap entire channel bank from top-of-bank to below depth of anticipated scour. Scour depth evaluated as illustrated in Section 17.6.7.2:

$d_s = 6.5\ D_{50}^{-0.11}$ (Equation 17.3)
$d_s = 6.5\ (0.5)^{-0.11} = 7.0$ ft

Project Example 2

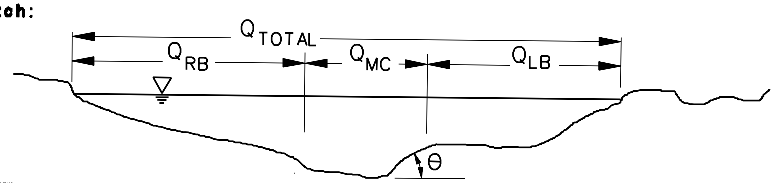
Description

Prepared by/date ______ / ______

Checked by/date ______ / ______

sheet ______ of ______

Definition Sketch:



Q_{TOTAL} ______

Q_{MC} ______

Q_{LB} ______

Q_{RB} ______

Soil Characteristics:

D_{15} ______

D_{50} ______

D_{85} ______

DEPTH OR W.S. (ft) ①	A (ft^2) ②	V_a (ft/sec) ③	d_a (ft) ④	Θ ⑤	Φ ⑥	K_1 ⑦	D_{50} (ft) ⑧	SF ⑨	S_s ⑩	C ⑪	$C_{P/A}$ ⑫	D_{50} (ft) ⑬	NOTES ⑭
15	2750	12.6	12	2:1	41°	0.73	0.9	1.6	2.6	1.6	1	1.44	Sharp Bend

Design Sketch

RIPRAP CHARACTERISTICS:

Size:

D_{50} 1.8 ft

Class 1/4 Ton

Thickness:

$2D_{50}$ 3.6 ft

D_{100} 2.25 ft

Use 3.6 ft

AASHTO Gradation:

Size: (ft)	Percent Finer
2.25	100
1.8	50
0.95	5-10

FABRIC CHARACTERISTICS:

Granular:

Size (ft)	Percent Finer
	85
	50
	15

Fabric:

AOS * < ______

Perm.> ______

* Average Opening Size

① Water Surface Elevation
② Main channel flow area
③ Main channel average velocity
④ Main channel average depth
⑤ Bank angle
⑥ Riprap angle of repose (Chart 4)
⑦ Bank angle correction (Chart 3)
⑧ Riprap size (Chart 1)
⑨ Stability Factor
⑩ Riprap specific gravity
⑪ Riprap size correction factor (Chart 2)
⑫ Pier/Abutment Correction (3.38 if applicable)
⑬ Correction D_{50} = ⑧ · ⑪ · ⑫
⑭ Notes or Comments

FIGURE 17-23 — Riprap Size Form (Example 2)

Adding this to the observed maximum depth yields a potential maximum scour depth of (below design water surface elevation):

15.0 + 7.0 = 22.0 ft

The bank material should be run to this depth, or a sufficient volume of stone should be placed at the bank toe to protect against the necessary depth of scour.

Step 13 Filter Layer Design:

(a) Filter material size (Figure 17-11):

$$\frac{D_{15}\ [\text{coarser layer}]}{D_{15}\ [\text{finer layer}]} < 5 < \frac{D_{15}\ [\text{coarser layer}]}{D_{15}\ [\text{finer layer}]} < 40$$

For the riprap to soil interface:

$$\frac{D_{15}\ [\text{riprap}]}{D_{85}\ [\text{soil}]} = \frac{0.5}{0.0042} = 119 > 5, \text{ and}$$

$$\frac{D_{15}\ [\text{riprap}]}{D_{15}\ [\text{soil}]} = \frac{0.5}{0.0042} = 119 > 40$$

Therefore, a filter layer is needed. Try ½ in uniformly graded fine gravel filter (gradation characteristics as illustrated in Figure 17-10). For the filter to soil interface:

$$\frac{D_{15}\ [\text{filter}]}{D_{85}\ [\text{soil}]} = \frac{0.015}{0.0042} = 3.6 < 5, \text{ and}$$

$$\frac{D_{15}\ [\text{filter}]}{D_{15}\ [\text{soil}]} = \frac{0.015}{0.00045} = 33 > 5 \text{ and } < 40$$

Therefore, filter to soil interface is OK. For the riprap to filter interface:

$$\frac{D_{15}\ [\text{riprap}]}{D_{85}\ [\text{filter}]} = \frac{0.5}{0.10} = 5 \le 5, \text{ and}$$

$$\frac{D_{15}\ [\text{riprap}]}{D_{15}\ [\text{filter}]} = \frac{0.5}{0.015} = 33.3 > 5 \text{ and } < 40$$

Therefore, the 1/2 in filter material is adequate.

(b) Filter layer thickness: Because soil gradation curve and filter layer, riprap and bank soil are approximately parallel, use layer thickness of 8 in.

Step 14 Edge Details:

(a) Flank details: See Figure 17-22.

(b) Toe detail: See Figure 17-22.

Anticipated scour depth below existing channel bottom at the bank (d'_s) is the depth of scour (computed in Step 12) minus the current bed elevation at the bank (see Figure 17-21): 22 ft – 12 ft = 10 ft.

Rock quantity required below the existing bed:

$$R_q = d'_s(\sin^{-1}\theta)(T)(1.5) \tag{17.12}$$

where:
- R_q = required riprap quantity per foot of bank, ft^2
- θ = the bank angle with the horizontal, degrees
- T = the riprap layer thickness, ft

$$R_q = (10)(2.24)(3)(1.5) = 101 \text{ ft}^2$$

A 6-ft deep trapezoidal toe trench with side slopes of 1V:2H and 1V:1H, and a bottom width of 6 ft, contains the necessary volume. Figure 17-22 illustrates the resulting toe trench detail.

17.7.2 Wire-Enclosed Rock

As described in Section 17.5.3, wire-enclosed rock (gabion) revetments consist of rectangular wire-mesh baskets filled with rock. The most common types of wire-enclosed revetments are mattresses and stacked blocks. The wire baskets that make up the mattresses and blocks are available from commercial manufacturers. If desired, these wire baskets can be fabricated from wire fencing materials.

Rock and wire mattress revetments consist of flat wire baskets or units filled with rock that are laid end to end and side to side on a prepared channel bed and/or bank. The individual mattress units are wired together to form a continuous revetment mattress.

Stacked block gabion revetments consist of rectangular wire baskets that are filled with stone and stacked in a stepped-back fashion to form the revetment surface. They are also commonly used at the toe of embankment slopes as toe walls that help to support other upper bank revetments and prevent undermining.

The rectangular basket or gabion units used for stacked configurations are of more uniform dimensions than those typically used for mattress designs. That is, they typically have a square cross section. Commercially available gabions used in stacked configurations are available in various sizes but the most common have a 3-ft width and thickness. Conceptually, the gabion units for stacked-block configurations could be fabricated from available fencing materials. However, the labor-intensive nature of such an installation makes it impractical in most cases. Therefore, only commercially available units are considered in the following sections.

17.7.2.1 Design Guidelines for Mattresses

Components of a rock and wire mattress design include layout of a general scheme or concept, bank and foundation preparation, mattress size and configuration, stone size, stone quality, basket or rock enclosure fabrication, edge treatment and filter design. Design guidance is provided below in each of these areas.

17.7.2.1.1 General

Rock and wire mattress revetments can be constructed from commercially available wire units as illustrated in the details of Figures 17-24 and 17-25 or from available wire fencing material as illustrated in Figure 17-26. The use of commercially available basket units is the most common practice and usually the least expensive.

Rock and wire mattress revetments can be used to protect either the channel bank (as illustrated in the section of Figure 17-24) or the entire channel perimeter (Figure 17-25). When used for bank protection, rock and wire mattress revetments consist of two distinct sections — a toe section and upper-bank paving (see Figure 17-24). As illustrated in Figure 17-24, a variety of toe designs can be used. Emphasis in design should be placed on toe design and filter design. These designs are detailed later.

The vertical and longitudinal extent of the mattress should be based on guidelines provided in Section 17.6.7. Emphasis in design should be placed on toe design and filter design.

17.7.2.1.2 Bank and Foundation Preparation

Channel banks should be graded to a uniform slope. The graded surface, either on the slope or on the stream bed at the toe of the slope on which the rock and wire mattress is to be constructed, should not deviate from the specified slope line by more than 6 in. All blunt or sharp objects (e.g., rocks, tree roots) protruding from the graded surface should be removed.

17.7.2.1.3 Mattress Unit Size and Configuration

Individual mattress units should be a size that is easily handled on site. Commercially available gabion units come in standard sizes as indicated in Table 17-6. Manufacturer's literature indicates that alternative sizes can be manufactured when required, provided that the quantities involved are of a reasonable magnitude. The mattress should be divided into compartments so that failure of one section of the mattress will not cause loss of the entire mattress. Compartmentalization also adds to the structural integrity of individual gabion units. It is recommended that diaphragms be installed at a nominal spacing of 3 ft within each of the gabion units to provide the recommended compartmentalization (see Figure 17-27).

On steep slopes greater than 3V:1H and in environments subject to high stresses (in areas prone to high-flow velocities, debris flows or ice flows), diaphragms should be spaced at minimum intervals of 2 ft to prevent movement of the stone inside the basket.

The thickness of the mattress is determined by three factors — the erodibility of the bank soil, the maximum velocity of the water and the bank slope. The minimum thickness required for various conditions is tabulated in Table 17-7. These values are based on observations of a large number of mattress installations that assume a filling material in the size range of 3 in to 6 in.

The mattress thickness should be at least as thick as two overlapping layers of stone. The thickness of mattresses used as bank toe aprons should always exceed 12 in. The typical range is 12 in to 20 in. The thickness of mattress revetments can vary according to need by utilizing gabions of different depths as illustrated in Figure 17-24(d).

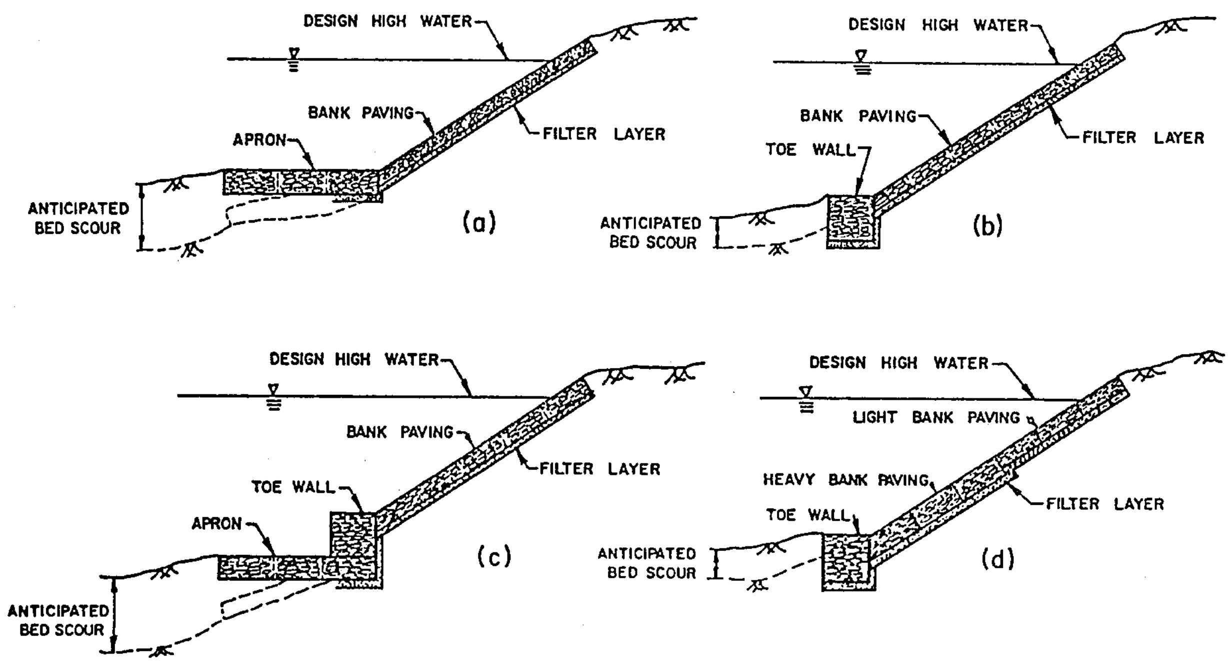


FIGURE 17-24 — Rock and Wire Mattress Configurations:
(a) mattress with toe apron; (b) mattress with toe wall; (c) mattress with toe wall and apron; and (d) mattress of variable thickness with toe

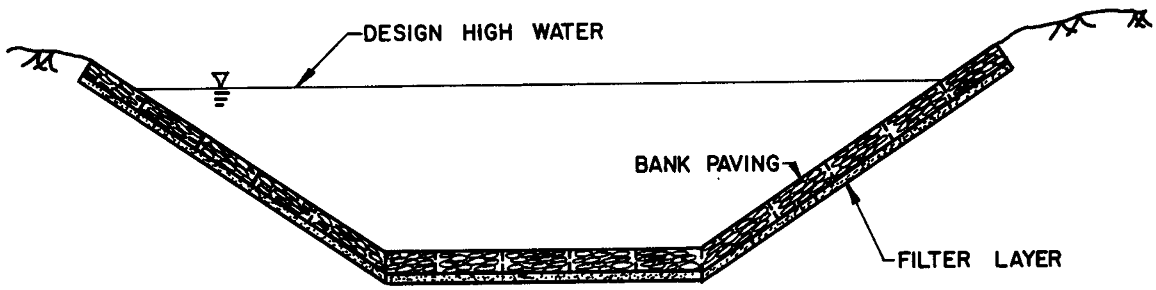


FIGURE 17-25 — Rock and Wire Mattress Installation Covering the Entire Channel Perimeter

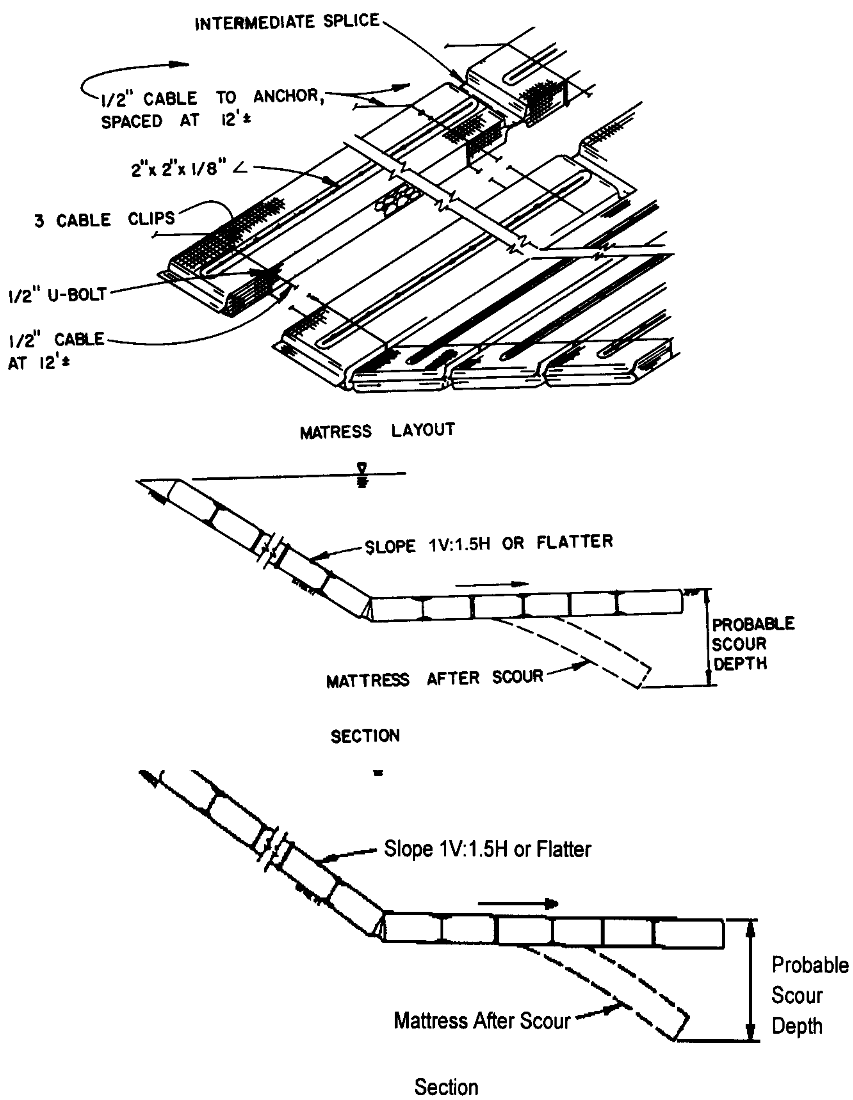


FIGURE 17-26 — Typical Detail of Rock, and Wire Mattress Constructed From Available Wire Fencing Materials

TABLE 17-6 — Standard Gabion Sizes

Thickness (ft)	Width (ft)	Length (ft)	Wire-Mesh Opening Size (in x in)
0.75	6	9	3 x 3
0.75	6	12	3 x 3
1.0	3	6	3 x 3
1.0	3	9	3 x 3
1.5	3	12	3 x 3
1.5	3	6	3 x 3
1.5	3	9	3 x 3
1.5	3	12	3 x 3
3.0	3	6	3 x 3
3.0	3	9	3 x 3
3.0	3	12	3 x 3

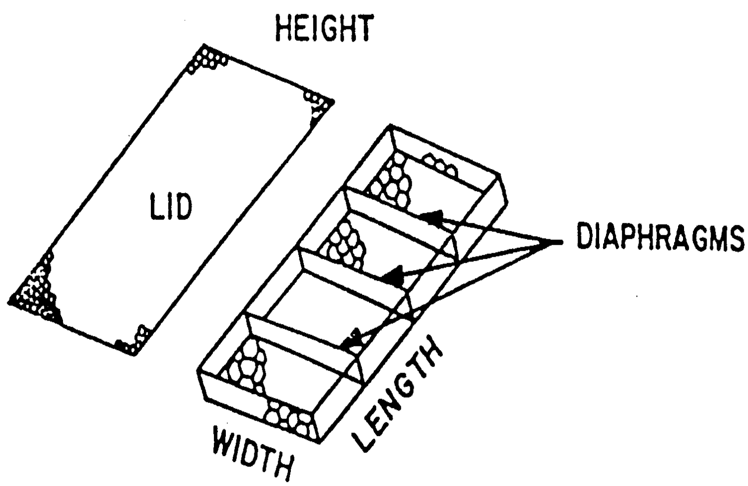


FIGURE 17-27 — Mattress Configuration

TABLE 17-7 — Criteria for Gabion Thickness

Bank Soil Type	Maximum Velocity (ft/s)	Bank Slope (V:H)	Minimum Required Mattress Thickness (in)
Clays, heavy Cohesive Soils	10 13 – 16 any	< 3:1 < 2:1 > 2:1	9 12 ≥ 18
Silts, fine sands	10	< 2:1	12
Shingle with gravel	16 20 any	< 3:1 < 2:1 > 2:1	9 12 ≥ 18

17.7.2.1.4 Stone Size

The maximum size of stone should be as specified. It must have suitable compressive strength and durability to resist the loading, effects of water and weathering. Usually, 0.3 ft to 1.0 ft clean stone is appropriate. A well-graded fill increases density. Place the stone evenly by hand to minimize voids and to control appearance along exposed faces.

17.7.2.1.5 Stone Quality

The stone should meet the quality requirements as specified for dumped-rock riprap.

17.7.2.1.6 Basket Fabrication

Commercially fabricated basket units are formed from galvanized steel wire mesh of triple twist hexagonal weave. The netting wire and binding wire is approximately No. 13½ gage. The wire for edges and corners is approximately No. 12 gage. Manufacturer's instructions for field assembly of basket units should be followed.

All wire used in the construction of the mesh rock enclosures (including tie wire) shall be zinc coated (galvanized) to A641M, Class 3 or aluminized coated to ASTM A809; the minimum weight of the zinc coating shall be as in Table 17-8.

TABLE 17-8 — Minimum Coating Weight (ASTM A641M, Class 3)

Gage	Nominal Diameter of Wire (in)	Minimum Coating Weight (oz/ft^2)
13½	0.086	0.7
12	0.104	0.8
10	0.128	0.9

Galvanized wire baskets may be safely used in fresh water and in areas where the pH of the liquid in contact with it is not greater than 10.

For highly corrosive conditions (e.g., in salt-water environments, industrial areas, polluted streams) and in soils such as muck, peat and cinders, a polyvinyl chloride (PVC) coating must be used over the galvanizing. The PVC coating must have a nominal thickness of 0.02165 in and shall nowhere be less than 0.015 in. It shall be capable of resisting deleterious effects of natural weather exposure and immersion in salt water and shall not show any material difference in its initial characteristics with time.

17.7.2.1.7 Edge Treatment

The edges of rock and wire mattress revetment installations (the toe, head and flanks) require special treatment to prevent damage from undermining. Of primary concern is toe treatment. Figure 17-24 illustrates several possible toe configurations. If a toe apron is used, its projection should be 1.5 times the expected maximum depth of scour in the vicinity of the revetment toe. In areas where little toe scour is expected, the apron can be replaced by a single-course gabion toe wall that helps to support the revetment and prevent undermining. Where an excessive amount of toe scour is anticipated, both an apron and a toe wall can be used.

To provide extra strength at the revetment flanks, it is recommended that mattress units having additional thickness be used at the upstream and downstream edges of the revetment (see

Figure 17-28). It is further recommended that a thin layer of topsoil be spread over the flank units to form a soil layer to be seeded when the revetment installation is complete. The head of rock and wire mattress revetments usually can be terminated at-grade as illustrated in Figure 17-24.

17.7.2.1.8 Filter Design

Individual mattress units will act as a crude filter and a pavement unit when filled with overlapping layers of hand-size stones; however, it is recommended that the need for a filter be investigated. If necessary, a layer of permeable membrane cloth (geotextile) woven from synthetic fibers, or a 4-in to 6-in layer of gravel, should be placed between the silty bank and the rock and wire mattress revetment to further inhibit washout of fines.

17.7.2.1.9 Construction

Construction details for rock and wire mattresses vary with the design and purpose for which the protection is provided. Typical details are illustrated in Figures 17-24 through 17-26. Rock and wire mattress revetments may be fabricated where they are to be placed or at an off-site location. Fabrication at an off-site location requires that the individual mattress units be transported to the site. In this case, extreme care must be taken so that moving and placing the baskets does not damage them by breaking or loosening strands of wire or ties or by removing any of the galvanizing or PVC coating. Because of the potential for damage to the wire enclosures, off-site fabrication is not recommended.

On-site fabrication of rock and wire mattress revetments is the most common practice. Figure 17-26 illustrates details for a rock and wire mattress constructed from galvanized fencing components. Figures 17-24 and 17-26 illustrate installations on a channel bank. Figure 17-25 illustrates a similar installation where the entire channel perimeter is lined.

Installation of mattress units above the water line is usually accomplished by placing individual units on the prepared bank, lacing them together, filling them with appropriately sized rock and then lacing the tops to the individual units. A typical installation is illustrated in Figure 17-29. Where the mattress units must be placed below the water line in relatively shallow water, mattress units can be assembled at a convenient location and then be placed on the bank using a crane as illustrated in Figure 17-30. For deep-water installations, an efficient method of large-scale placement is to fabricate the mattress sections on a barge or pontoon and then launch them into the water at the shoreline (see Figure 17-31).

17.7.2.2 Design Guidelines for Stacked-Block Gabions

Components of stacked-block gabion revetment design include layout of a general scheme or concept, bank and foundation preparation, unit size and configuration, stone size and quality, edge treatment, backfill and filter considerations, and basket or rock enclosure fabrication. Design guidelines for stone size and quality and bank preparation are the same as those discussed for mattress designs; other remaining areas are discussed below.

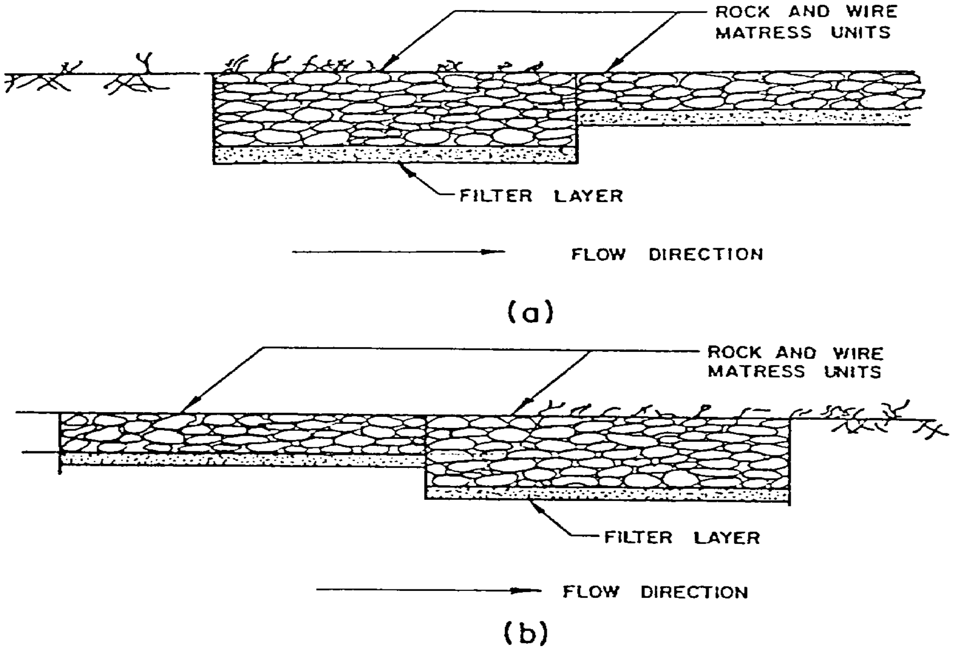


FIGURE 17-28 — Flank Treatment for Rock and Wire Mattress Designs: (a) upstream face; (b) downstream face

FIGURE 17-29 — Rock and Wire Revetment Mattress Installation

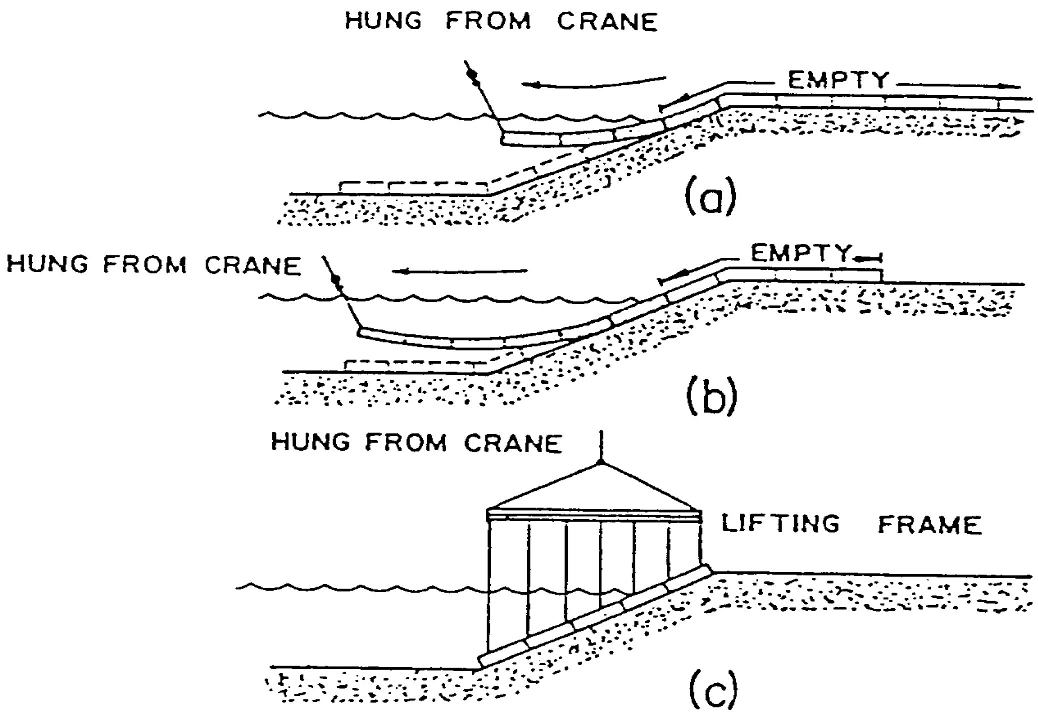


FIGURE 17-30 — Mattress Placement Underwater by Crane

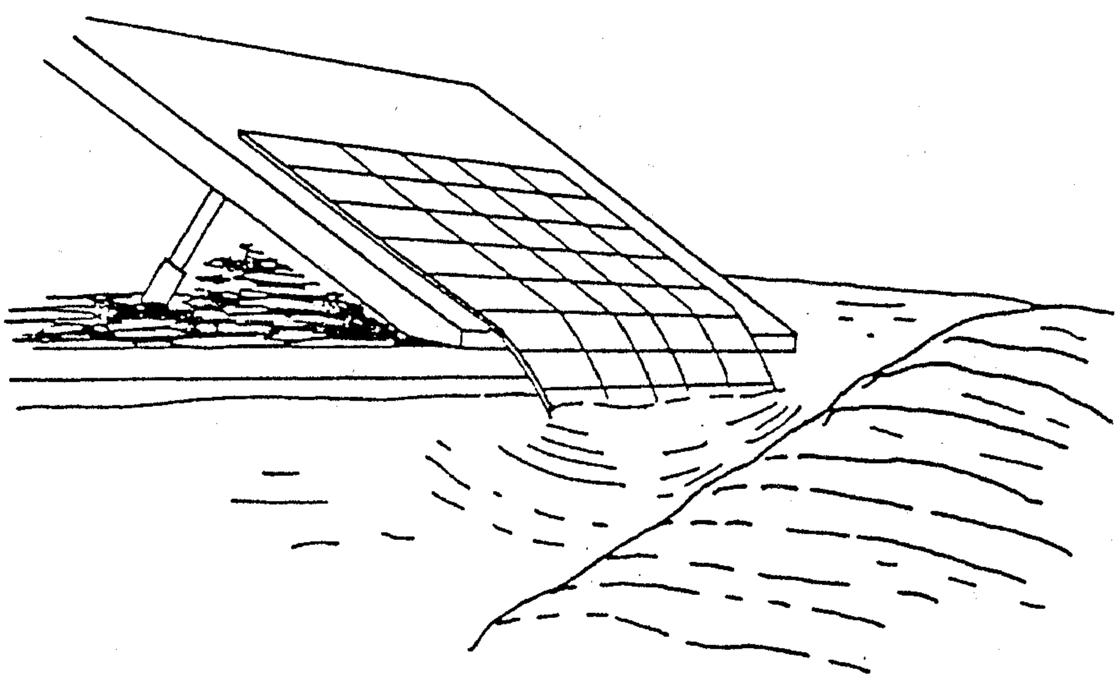


FIGURE 17-31 — Pontoon Placement of Wire Mattress

17.7.2.2.1

General

Stacked-gabion revetments are used instead of gabion mattress designs where the slope to be protected is greater than 1V:1H or where the purpose of the revetment is for flow training. They can be used as retaining structures where space limitations prohibit bank grading to a slope suitable for other revetments. Typical design schemes include flow training walls, Figure 17-32(a), and low or high retaining walls, Figures 17-32(b and c), respectively.

Stacked-gabion revetments must be based on a firm foundation. The foundation or base elevation of the structure should be well below any anticipated scour depth. Additionally, in alluvial streams where channel bed fluctuations are common, an apron should be used as illustrated in Figure 17-32(a and b). Aprons are recommended where the estimated scour depth is uncertain.

17.7.2.2.2 Size and Configuration

Common commercial sizes for stacked gabions are listed in Table 17-6. The most common sizes have widths and depths of 3 ft. Sizes less than 1 ft thick are not practical for stacked-gabion installations.

Typical design configurations include flow training walls and structural retaining walls. The primary function of flow training walls (Figure 17-32(a)) is to establish normal channel boundaries in rivers where erosion has created a wide channel or to realign the river where it is encroaching on an existing or proposed structure. A stepped-back wall is constructed at the desired bank location; counterforts are installed to tie the walls to the channel bank at regular intervals and as illustrated. The counterforts are installed to form a structural tie between the training wall and the natural stream bank and to prevent overflow from scouring a channel behind the wall. Counterforts should be spaced to eliminate the development of eddy or other flow currents between the training wall and the bank. The eddy or flow currents could cause further erosion of the bank. The dead water zones created by the counterforts, spaced in this manner, will encourage sediment deposition behind the wall that will enhance the stabilizing characteristics of the wall.

Retaining walls can be designed in either a stepped-back configuration as illustrated in Figures 17-32(b) and (c) or a batter configuration as illustrated in Figure 17-32(d). Structural details and configurations can vary from site to site.

Gabion walls are gravity structures, and their design follows standard engineering practice for retaining structures. Design procedures are available in standard soil mechanics texts and in gabion manufacturer’s literature.

17.7.2.2.3 Edge Treatment

The flanks and toe of stacked-block gabion revetments require special attention. The upstream and downstream flanks of these revetments should include counterforts; see Figure 17-32(a). The counterforts should be placed 12 ft to 18 ft from the upstream and downstream limits of the structure and should extend a minimum of 12 ft into the bank.

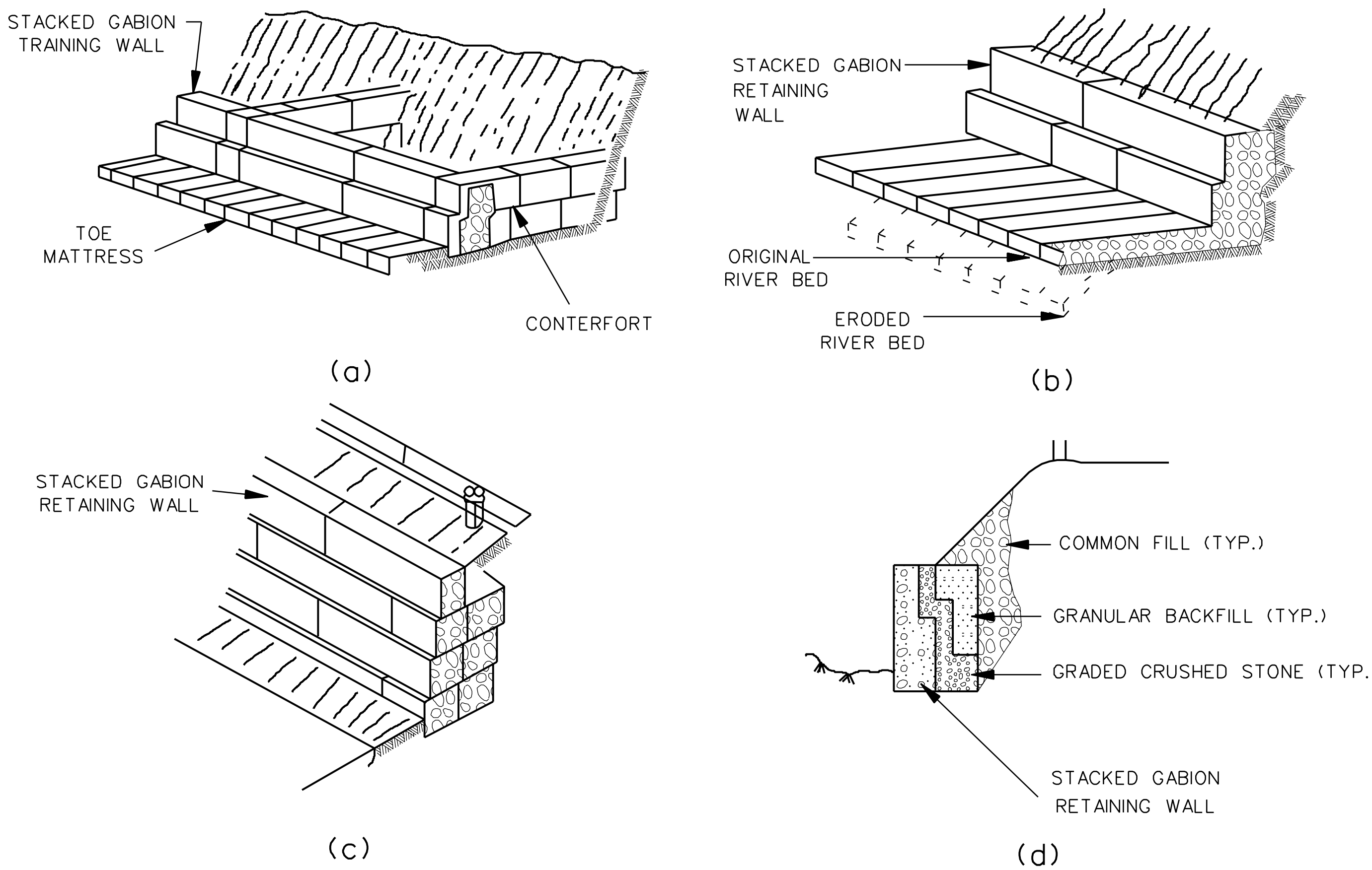


FIGURE 17-32 — Typical Stacked-Block Gabion Revetment Details: (a) training wall with counterforts; (b) stepped-back low retaining wall with apron; (c) high retaining wall, stepped-back configuration; (d) high retaining wall, batter type

The toe of the revetment should be protected by placing the base of the gabion wall at a depth below anticipated scour depths. Where it is difficult to predict the depth of expected scour or where channel bed fluctuations are common, it is recommended that a mattress apron be used. The minimum apron length should be equal to 1.5 times the anticipated scour depth below the apron. This length can be increased in proportion to the level of uncertainty in predicting the local toe scour depth.

17.7.2.2.4 Backfill/Filter Requirements

Standard retaining wall design requires the use of selected backfill behind the retaining structure to provide for drainage of the soil mass behind the wall. The permeable nature of gabion structures permits natural drainage of the supported embankment. However, because material leaching through the gabion wall can become trapped and cause plugging, it is recommended that a granular backfill material be used (see Figure 17-32(d)). The backfill should consist of a 2-in to 12-in layer of graded crushed stone backed by a layer of fine granular backfill.

17.7.2.2.5 Basket Fabrication

Commercially fabricated basket units are formed from galvanized steel wire mesh of triple twist hexagonal weave. The netting wire and binding wire specifications are the same as those discussed for mattress units. Specifications for galvanizing and PVC coatings are also the same for block designs as for mattresses. Figure 17-33 illustrates typical details of basket fabrication.

17.7.2.2.6 Construction

Construction details for gabion installations typically vary with the design and purpose for which the protection is being provided. Several typical design schematics are presented in Figures 17-32 and 17-33. Design details for a typical stepped-back design and a typical batter design are presented in Figure 17-34.

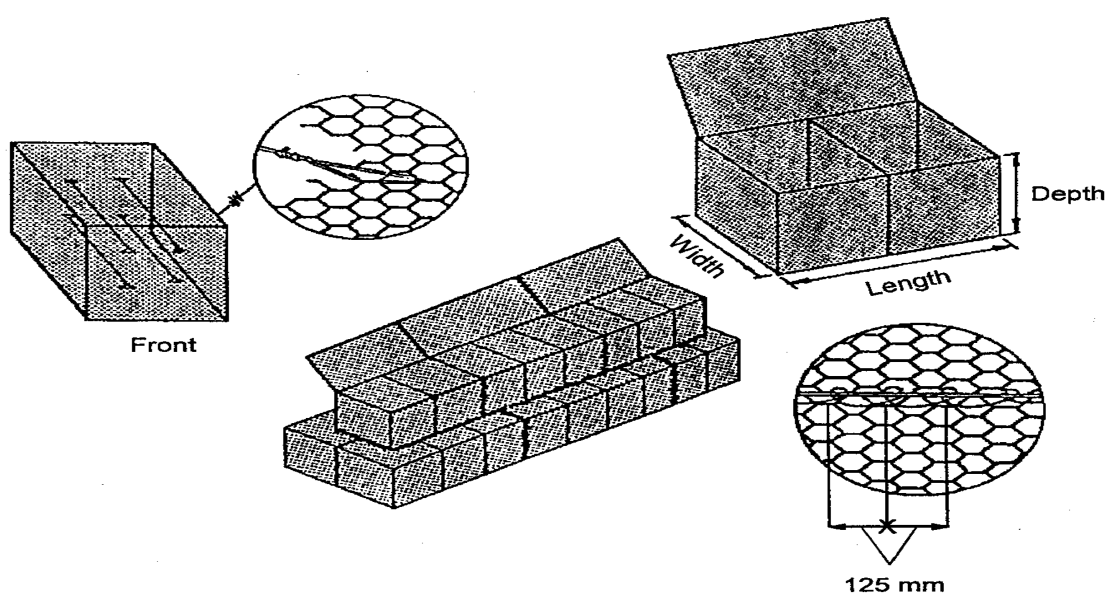
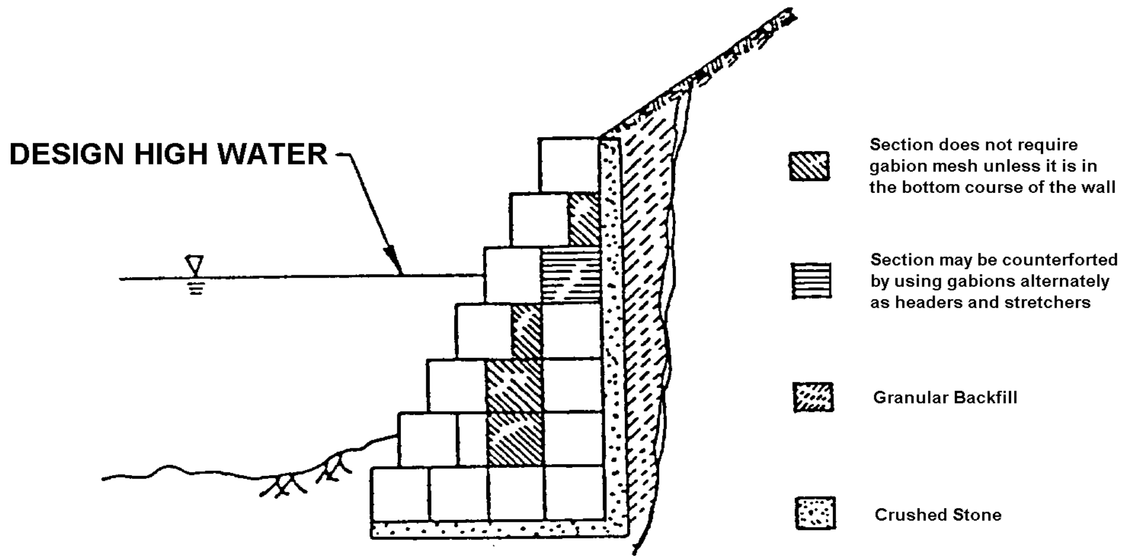
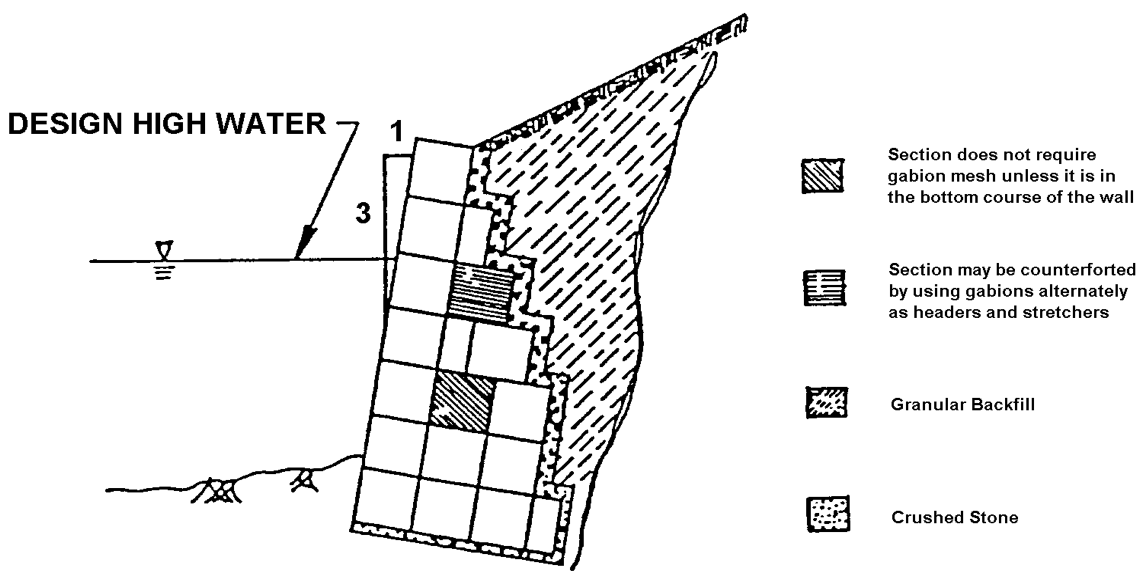


FIGURE 17-33 — Gabion Basket Fabrication



(a)



(b)

FIGURE 17-34 — Section Details: (a) stepped-back and (b) battered-gabion retaining walls

As with mattress designs, fabrication and filling of individual basket units can be done at the site or at an off-site location. The most common practice is to fabricate and fill individual gabions at the construction site. The following Steps outline the typical sequence used for installing a stacked-gabion revetment or wall:

Step 1 Prepare the revetment foundation. This includes excavation for the foundation and revetment wall.

Step 2 Place the filter and gabion mattress (for designs that incorporate this component) on the prepared grade, then sequentially stack the gabion baskets to form the revetment system.

Step 3 Each basket is unfolded and assembled by lacing the edges together and the diaphragms to the sides.

Step 4 Fill the gabions to a depth of 1 ft with stone from 4 in to 12 in in diameter. Place one connecting wire in each direction and loop it around two meshes of the gabion wall. Repeat this operation until the gabion is filled.

Step 5 Wire adjoining gabions together by their vertical edges; stack empty gabions on the filled gabions and wire them at front and back.

Step 6 After the gabion is filled, fold the top shut and wire it to the ends, sides and diaphragms.

Step 7 Crushed stone and granular backfill should be placed at intervals to help support the wall structure. It is recommended that backfill be placed at three-course intervals.

17.7.3 Precast Concrete

Precast concrete block revetments consist of preformed sections that interlock with each other, are attached to each other or butt together to form a continuous blanket or mat. The concrete blocks that make up the mats differ in shape and method of articulation but share certain common features. These features include flexibility, rapid installation and provisions for the establishment of vegetation within the revetment.

17.7.3.1 Block Designs

Precast concrete block designs come in a number of shapes and configurations. Figures 17-35 through 17-39 illustrate several commercially available concrete block designs. Note that other manufacturers and designs are available. Precast revetments are bound together using a variety of techniques. In some cases, the individual blocks are bound to rectangular sheets of filter fabric (referred to as fabric carrier). Other manufacturers use a design that interlocks individual blocks. Other units are simply butted together at the site. The most common method is to join individual blocks with wire cable or synthetic fiber rope. See References *(5)*, *(6)* for more information.

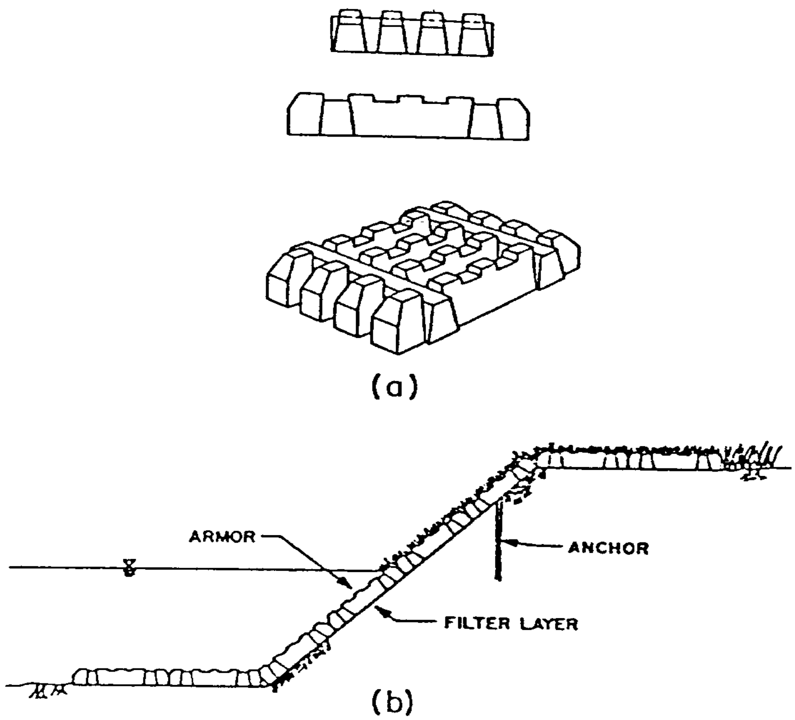


FIGURE 17-35 — Monoslab Revetment (a) block detail and (b) revetment detail

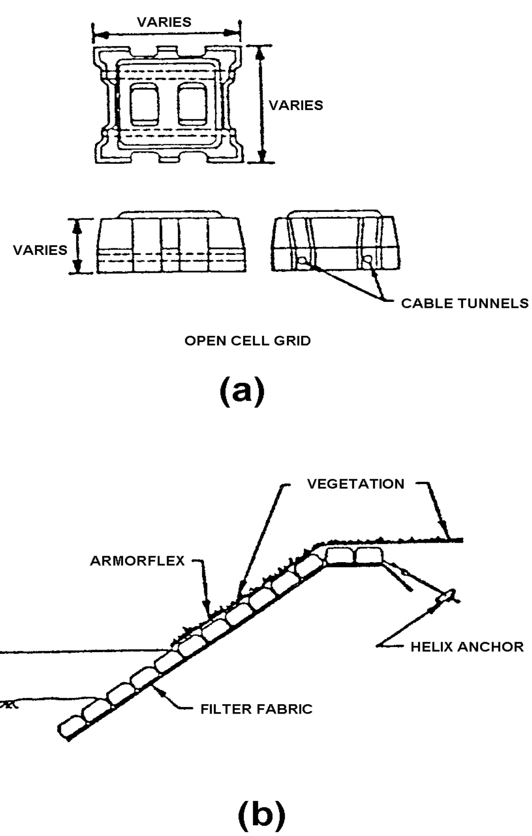


FIGURE 17-36 — Armorflex (a) block detail and (b) revetment configuration

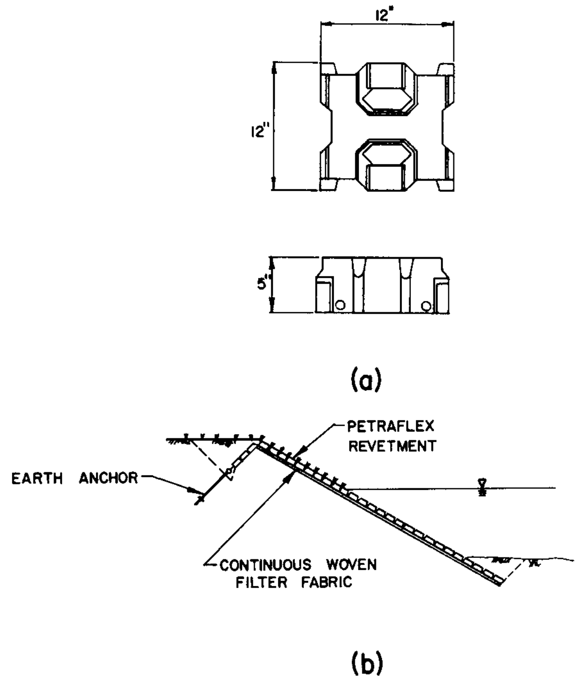


FIGURE 17-37 — Petraflex (a) block detail and (b) revetment configuration

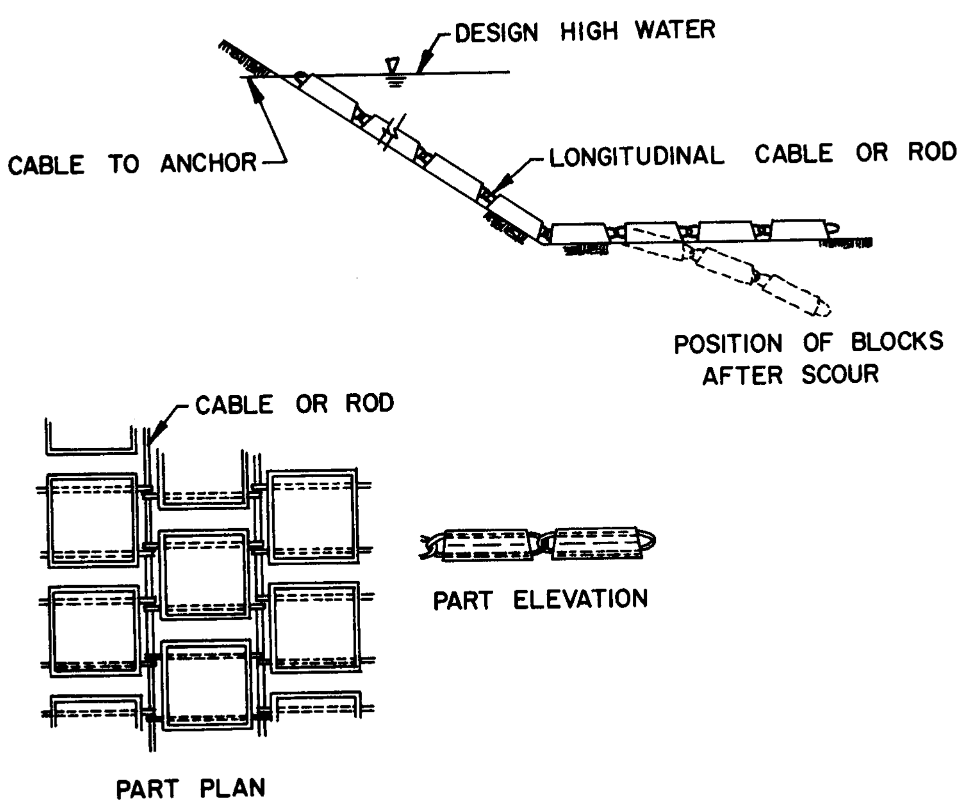
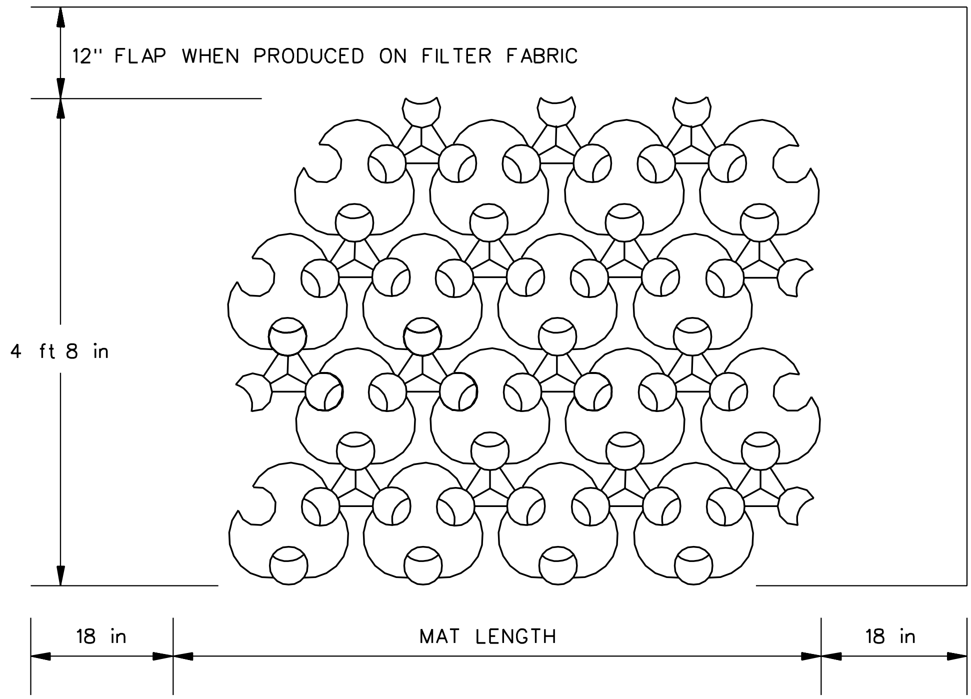


FIGURE 17-38 — Articulated Concrete Revetment



Note: Dimensions may vary.

FIGURE 17-39 — Tri-lock Revetment

17.7.3.2 Design Guidelines

Components of a precast concrete block revetment design include the layout of a general scheme or concept, bank preparation, mattress and block size, slope, edge treatment, filter design and surface treatment. Design information is provided below in each of these areas.

As illustrated in Figure 17-35 through Figure 17-39, precast block revetments are placed on the channel bank as continuous mattresses. Emphasis in design should be placed on edge treatment, toe and filter design.

17.7.3.2.1 Bank Preparation

Channel banks should be graded to a uniform slope. Any large boulders, roots and debris should be removed from the bank prior to final grading. Also, holes, soft areas and large cavities should be filled. The graded surface, either on the slope or on the stream bed at the toe of the slope on which the revetment is to be constructed should be true to line and grade. Light compaction of the bank surface is recommended to provide a solid foundation for the mattress.

17.7.3.2.2 Mattress and Block Size

The overall mattress size is dictated by the longitudinal and vertical extent required of the revetment system. Articulated block mattresses are assembled in sections prior to placement on the bank; individual mattresses should be constructed to a size that is easily handled on site by available construction equipment. The size of individual blocks is quite variable from manufacturer to manufacturer. In addition, individual manufacturers usually have several standard sizes of a particular block available. Manufacturer's literature should be consulted when selecting an appropriate block size for a given hydraulic condition.

17.7.3.2.3 Slope

Articulated precast block revetments can be used on bank slopes up to 1V:1.5H. However, an earth anchor should be used at the top of the revetment to secure the system against slippage (see Figures 17-35 and 17-37). Precast block revetments that are assembled by simply butting individual blocks end to end (with no physical connection) should not be used on slopes greater than 1V:3H.

17.7.3.2.4 Edge Treatment

The edges of precast block revetments (the toe, head and flanks) require special treatment to prevent undermining. Of primary concern in the design of mattress revetments is the toe treatment. Two toe treatments have been used — an apron design, as illustrated in Figures 17-35 and 17-38, and a toe trench design, as illustrated in Figures 17-36 and 17-37. At a minimum, toe aprons should extend 1.5 times the anticipated scour depth in the vicinity of the bank toe. If a toe trench is used, the mattress should extend to a depth greater than the anticipated scour depth in the vicinity of the bank toe.

Two alternatives have been used for edge treatments at the top and flanks. The edges can be terminated at-grade (Figures 17-35, 17-36 and 17-38) or in a termination trench. Termination trenches are recommended in environments subject to significant erosion (silty/sandy soils and high velocities) or where failure of the revetment would result in significant economic loss. Termination trenches provide greater protection against failure from undermining and outflanking than do at-grade terminations; however, where upper-bank erosion or lateral outflanking is not expected to be a problem, grade terminations may provide an economic advantage.

For articulated designs, earth anchors should be placed at regular intervals along the top of the revetment (see Figures 17-36 and 17-37). Anchors are spaced based on soil type, mat size and the size of the anchors. See the manufacturer's literature for recommended spacings.

17.7.3.2.5 Filter

Prior to installing the mats, a geotextile filter fabric should be installed on the bank to prevent bank material from leaching through the openings in the mattress structure. Although a fabric filter is recommended, graded filter material can be used if it is properly designed and installed to prevent movement of the graded material through the protective mattress.

17.7.3.2.6 Surface Treatment

The spaces between and within individual blocks located above the low water line should be filled with earth and seeded so that natural vegetation can be established on the bank (see Figures 17-36 and 17-37). This treatment enhances both the structural stability of the embankment and its aesthetic qualities.

17.7.3.3 Construction

Schematics of the types of precast block revetments discussed above are provided in Figures 17-35 through 17-39. More detailed design sketches and information are available from individual manufacturers. Manufacturers can provide information on construction procedures. Some manufacturers offer on-site advice and assistance in the installation of their systems.

Articulated preformed block revetments can be installed by construction crews using conventional construction equipment wherever a dragline or crane can be maneuvered. Construction procedures for most preformed block revetments are similar. After all site preparation work is completed, construction follows the following sequence:

Step 1 Excavate toe, flank and upper-bank protection trenches as required.

Step 2 Place filter fabric and/or graded filter material on the prepared subgrade.

Step 3 Individual mats are then attached to a spreader bar and lifted with a crane or backhoe for placement on the embankment slope. Mats are placed side by side on the bank until the entire prepared surface is covered.

Step 4 Adjacent mats are secured to one another by fastening side connecting cables and end loops or by pouring side connecting keys.

Step 5 Optional anchors are placed at the top and flanks of the protection as required.

Step 6 Backfill is then spread over the mats (and into the open cells or spaces between cells) and into the anchor trenches. Anchor trenches are then compacted, and the general backfill should be seeded and fertilized according to local seasonal conditions.

Non-articulated block revetments (i.e., where the blocks are butted together instead of being physically attached) are constructed in a similar fashion, except that the individual blocks must be placed on the bank by hand, one at a time. This results in a much more labor-intensive installation procedure.

17.7.4 Grouted Rock

Grouted rock revetment consists of rock slope protection having voids filled with concrete grout to form a monolithic armor. See Section 17.5.5 for additional descriptive information and general performance characteristics for grouted rock. See also section 4.4.3 of HEC 23 *(2)*.

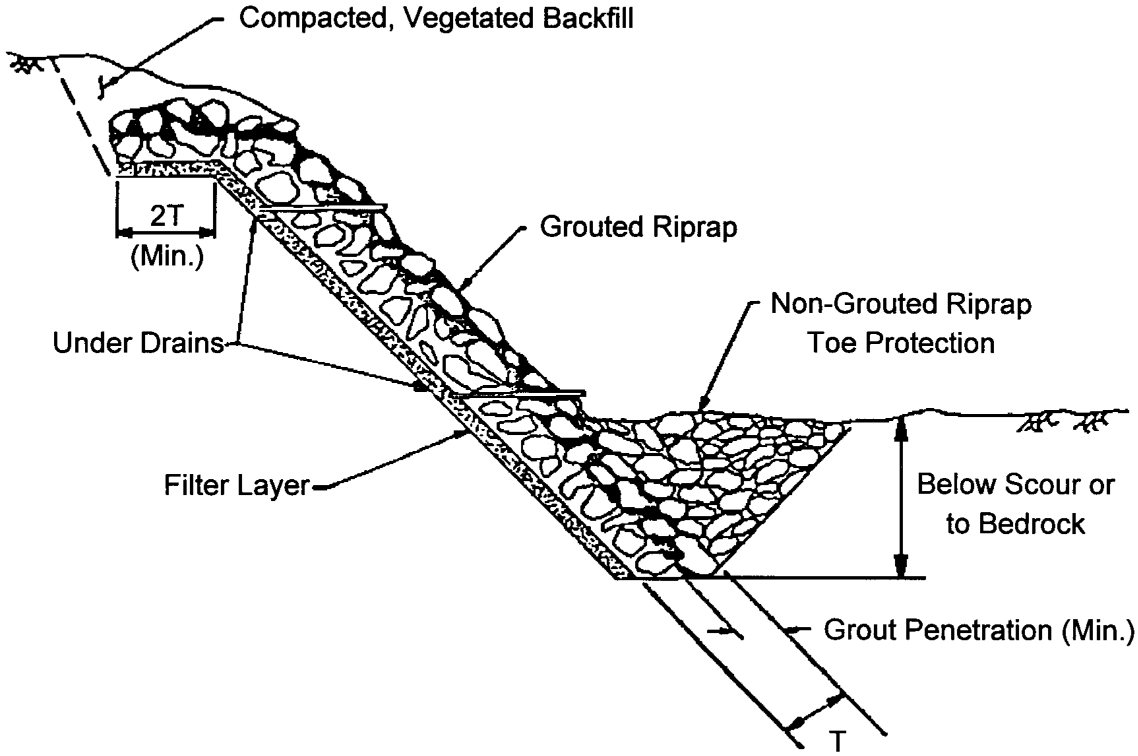
17.7.4.1 Design Guidelines

Components of grouted rock riprap design include layout of a general scheme or concept, bank preparation, bank slope, rock size and blanket thickness, rock grading, rock quality, grout quality, edge treatment, filter design and pressure relief.

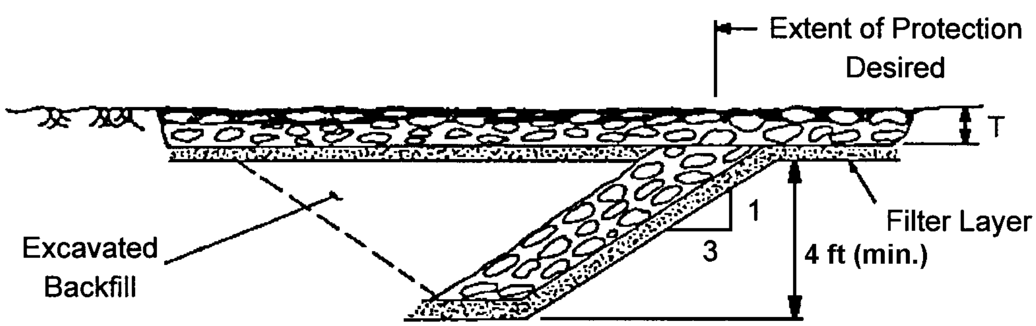
Grouted riprap designs are rigid monolithic bank protection schemes. When complete, they form a continuous surface. A typical grouted riprap section is shown in Figure 17-40.

Grouted riprap should extend from below the anticipated channel bed scour depth to the design high-water level, plus additional height for freeboard.

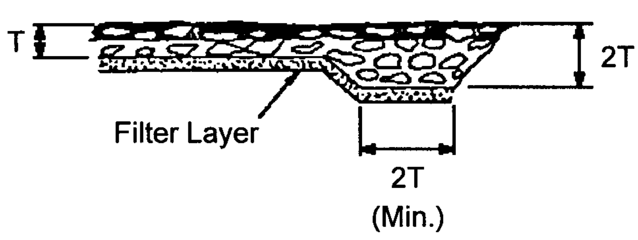
During the design phase for a grouted riprap revetment, special attention needs to be devoted to edge treatment, foundation design and mechanisms for hydrostatic pressure relief.



(a) Section A-A



(b) Section B-B



(c) Section C-C

FIGURE 17-40 — Grouted Riprap Sections: (a) Section A-A; (b) Section B-B; and (c) Section C-C

17.7.4.1.1

Bank and Foundation Preparation

The bank should be prepared by first clearing all trees and debris from the bank and grading the bank surface to the desired slope. In general, the graded surface should not deviate from the specified slope line by more than 6 in. However, local depressions larger than this can be accommodated because initial placement of filter material and/or rock for the revetment will fill these depressions.

Because grouted riprap is rigid but not strong, support by the embankment must be maintained. To form a firm foundation, it is recommended that the bank surface be tamped or lightly compacted. Care must be taken during bank compaction to maintain soil permeability similar to that of the natural, undisturbed bank material. The foundation for the grouted riprap revetment should have a bearing capacity sufficient to support either the dry weight of the revetment alone, or the submerged weight of the revetment plus the weight of the water in the wedge above the revetment for design conditions, whichever is greater.

17.7.4.1.2 Bank Slope

Bank slopes for grouted riprap revetments should not exceed 1V:2H.

17.7.4.1.3 Rock Size and Blanket Thickness

Blanket thickness and rock size requirements for a grouted-riprap installation are interrelated. Figure 17-41 illustrates a relationship between the design velocity and the required riprap blanket thickness for grouted-riprap designs. The median rock size in the revetment should not exceed 0.67 times the blanket thickness. The largest rock used in the revetment should not exceed the blanket thickness.

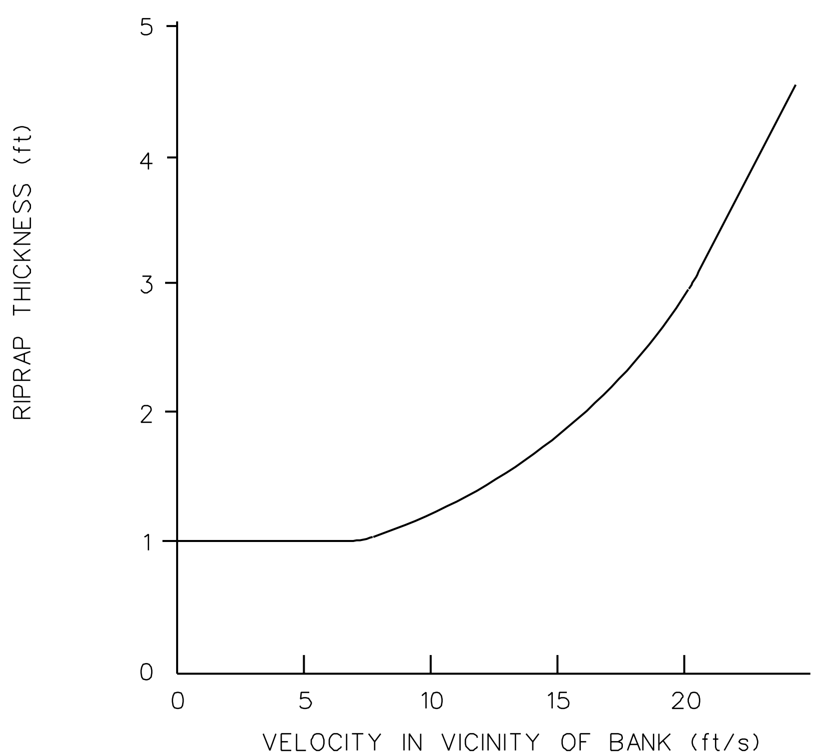


FIGURE 17-41 — Required Blanket Thickness as a Function of Flow Velocity

17.7.4.1.4 Rock Grading

Table 17-9 provides guidelines for rock gradation in grouted riprap installations. Six size classes are listed.

TABLE 17-9 — Recommended Grading of Grouted Rock Slope Protection

Rock Sizes		Classes (Percent larger than given rock size)					
Equivalent Diameter (ft)	Weight (Ton)	1 Ton	0.5 Ton	0.25 Ton	Light	Facing	Cobble
3.50	1.81	0-5					
2.75	1.81	50-100	0-5				
2.25	0.45	—	50-100	0-5			
1.75	0.23	95-100	—	50-100	0-5		
1.25	0.09		95-100	—	50-100	0-5	
1.00	0.03			95-100	95-100	50-100	0-5
0.50	0.01				95-100	95-100	95-100
Minimum Penetration of Grout (in)		24	18	14	10	8	6

17.7.4.1.5 Rock Quality

Rock used in grouted-rock slope protection is usually the same as that used in ordinary rock slope protection; however, the specifications for specific gravity and hardness may be lowered if necessary as the rocks are protected by the surrounding grout. The rock used in grouted-riprap installations should be free of fines so that penetration of grout may be achieved.

17.7.4.1.6 Grout Quality and Characteristics

Grout should consist of good strength concrete using a maximum aggregate size of 3/4 in and a slump of 3 in to 4 in. Sand mixes may be used where roughness of the grout surface is unnecessary, provided that sufficient cement is added to give good strength and workability.

The volume of grout required will be that necessary to provide penetration to the depths shown in Table 17-9. The finished grout should leave face stones exposed for one-fourth to one-third their depth, and the surface of the grout should expose a matrix of coarse aggregate.

17.7.4.1.7 Edge Treatment

The edges of grouted-rock revetments (the head, toe and flanks) require special treatment to prevent undermining. The revetment toe should extend to a depth below anticipated scour depths or to bedrock. The toe should be designed as illustrated in Figure 17-40(a). After excavating to the desired depth, the riprap slope protection should be extended to the bottom of the trench and grouted. The remainder of the excavated area in the toe trench should be filled with grout-free riprap. The grout-free riprap provides extra protection against undermining at the bank toe.

To prevent outflanking of the revetment, various edge treatments are required. Recommended designs for these edge treatments are illustrated in Figure 17-40 (a), (b) and (c).

17.7.4.1.8 Filter Design

Filters are required under all grouted riprap revetments to provide a zone of high permeability to carry off seepage water and prevent damage to the overlying structure from uplift pressures. A 6-in granular filter is required beneath the pavement to provide an adequate drainage zone. The filter can consist of well-graded granular material or uniformly graded granular material with an underlying filter fabric. The filter should be designed to provide a high degree of permeability while preventing base material particles from penetrating the filter, thus causing clogging and failure of the protective filter layer.

17.7.4.1.9 Pressure Relief

Weep holes should be provided in the revetment to relieve hydrostatic pressure buildup behind the grout surface (see Figure 17-40(a)). Weeps should extend through the grout surface to the interface with the gravel underdrain layer. Weeps should consist of 3-in diameter pipes having a maximum horizontal spacing of 6 ft and a maximum vertical spacing of 10 ft. The buried end of the weep should be covered with wire screening or a fabric filter of a gage that will prevent passage of the gravel underlayer.

17.7.4.2 Construction

Construction details for grouted riprap revetments are illustrated in Figure 17-40. The following construction procedures should be followed:

Step 1 Normal construction procedures include (a) bank clearing and grading; (b) development of foundations; (c) placement of the rock slope protection; (d) grouting of the interstices; (e) backfilling toe and flank trenches; and (f) vegetation of disturbed areas.

Step 2 The rock should be wet immediately prior to commencing the grouting operation.

Step 3 The grout may be transported to the place of final deposit by chutes, tubes, buckets, pneumatic equipment, or any other mechanical method that will control segregation and uniformity of the grout.

Step 4 Spading and rodding are necessary to ensure penetration of grout into the interstices.

Step 5 No loads should be allowed upon the revetment until good strength has been developed.

17.7.5 Concrete Slope Pavement

Concrete slope pavement revetments are cast in place or precast and set in place on a prepared slope to provide a continuous, monolithic armor for bank protection. Cast-in-place designs are the most common of the two design methods. For additional descriptive information and general performance characteristics of concrete pavement, see Section 17.5.6.

17.7.5.1 Design Guidelines

Components of concrete pavement revetment design include layout of a general scheme, bank and foundation preparation, bank slope, pavement thickness, pavement reinforcement, edge treatment, stub walls, filter design, pressure relief and concrete quality. Each of these components is addressed below.

Concrete pavement designs are ridged, monolithic, bank protection schemes. When complete, they form a continuous surface. A design sketch of a typical concrete pavement is illustrated in Figure 17-42. As illustrated in Figure 17-42, typical concrete pavement revetment consists of the bank pavement, a toe section, a head section, cutoff or stub walls, weeps and a filter layer.

Concrete pavements can be designed as light duty or heavy duty. The distinction between light and heavy duty concrete pavement is in the various dimensions labeled in Table 17-10.

As indicated in Figure 17-42, concrete pavements should extend vertically below the anticipated channel bed scour depth and to a height equal to the design high-water level plus additional height for freeboard. The longitudinal extent of protection should be as described in Section 17.6.7. One additional consideration in concrete pavement design is the surface texture. Depending on the smoothness required for hydraulics, a float or sand finish may be specified or, if roughness is desired, plans may call for surface roughening by raking the surface after the initial set.

During the design phase for concrete pavement revetment, special attention needs to be paid to toe and edge treatment, foundation design and mechanisms for hydrostatic pressure relief. Field experience indicates that inadequacies in these areas of design are often responsible for failures of concrete pavement revetments.

17.7.5.1.1 Bank and Foundation Preparation

The bank should be prepared by first clearing all trees and debris from the bank and grading the bank surface to a slope not to exceed 1V:2H.

Continuity of the final graded surface is important. After grading, the surface should be true to grade and stable with respect to slip and settlement. To form a firm foundation, it is recommended that the bank surface be tamped or lightly compacted. Care must be taken during bank compaction to maintain soil permeability similar to that of the natural, undisturbed bank material. After compaction, the bank surface should not deviate from the specified slope by more than 0.07 ft – 0.1 ft at any one point. This is particularly true if precast slabs are to be placed on the bank.

The foundation for the concrete slope pavement revetment should have a bearing capacity sufficient to support either the dry weight of the revetment alone, or the submerged weight of the revetment plus the weight of water in the wedge above the revetment for design conditions, whichever is greater.

17.7.5.1.2 Bank Slope

The bank slope for concrete pavements should not exceed 1V:1.5H.

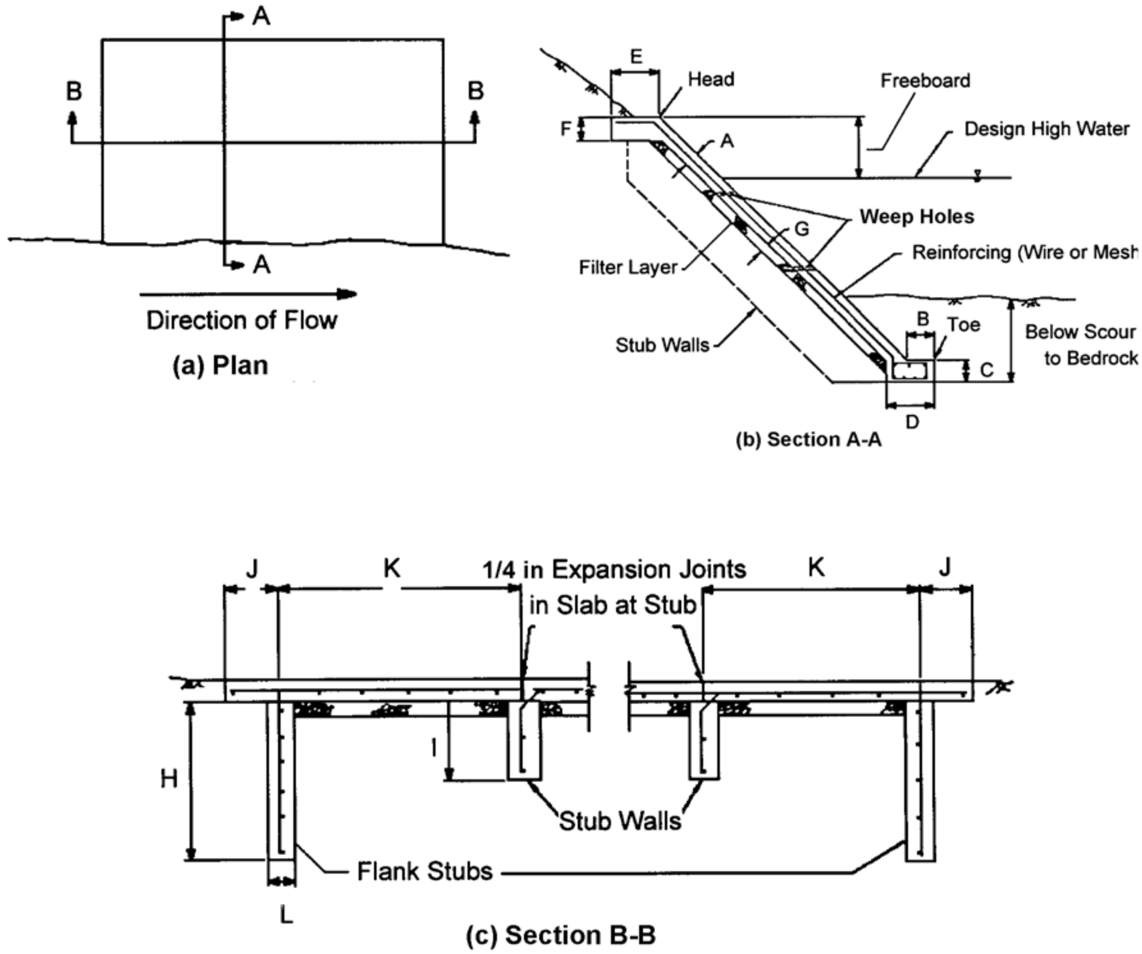


FIGURE 17-42 — Concrete Slope Paving Detail: (a) Plan; (b) Section A-A; (c) Section B-B

TABLE 17-10 — Dimensions for Concrete Slab

Revetment Class	Dimension (ft)											
	A	B	C	D	E	F	G	H	I	J	K	L
Light Duty	0.33	0	0.75	1.83	1.83	0.50	0.33	4 – 5	2 – 3	1.5	15 – 30	0.50
Heavy Duty	0.50	0.75	0.75	1.75	2.00	0.75	0.50	3.67 – 5	2 – 3	1.5	15 – 30	0.75

17.7.5.1.3 Pavement Thickness

A pavement thickness of 4 in to 6 in is recommended. Pavement thickness of up to 4 in is referred to as a light pavement and 6 in paving as heavy pavement.

17.7.5.1.4 Reinforcement

The purpose of reinforcement is to maintain the continuity of pavement even though cracks develop from shrinkage, thermal stresses and flexural stresses.

Reinforcement may be either mesh or bar reinforcement. Both size and spacing in each direction must be specified.

17.7.5.1.5 Concrete Quality

Concrete should be of good strength, and the concrete mixture shall be proportioned to secure a workable, finishable, durable, watertight and wear-resistant concrete of the desired strength. Class A (AASHTO classification) proportions are recommended; however, in some less critical design situations, Class B proportions may be used.

17.7.5.1.6 Edge Treatment

The edges of the concrete pavement (the toe, head and flanks) require special treatment to prevent undermining. Section A-A in Figure 17-42(b) illustrates standard head and toe designs. The head of the pavement should be tied into the bank and overlapped with soil as illustrated to form a smooth transition from the concrete pavement to the natural bank material. This minimizes scour due to the discontinuity in this area. Also, this design seals off the filter layer from any water that overtops the revetment, thereby reducing the potential for erosion at this interface.

Section A-A also illustrates the standard toe design. The revetment toe should extend to a depth below anticipated scour or to bedrock. Where this is not feasible without costly underwater construction, an alternative design should be considered. Several alternative designs are illustrated in Figure 17-43, including a riprap-filled toe trench, a toe mattress and a sheet-pile toe wall. In all but the latter case, the concrete pavement should extend a minimum of 5 ft below the channel bed; the sheet-pile toe wall can be attached to the concrete pavement above, below, or at the channel bed level.

Section B-B (Figure 17-42(c)) illustrates flank treatment. At the upstream and downstream limits, flank stubs are used to prevent progressive undermining at the flanks.

17.7.5.1.7 Stub Walls

As illustrated in Figure 17-42(c), stub walls should be placed at regular intervals. Stub walls provide support for the revetment at expansion joints; they also guard against progressive failure of the revetment.

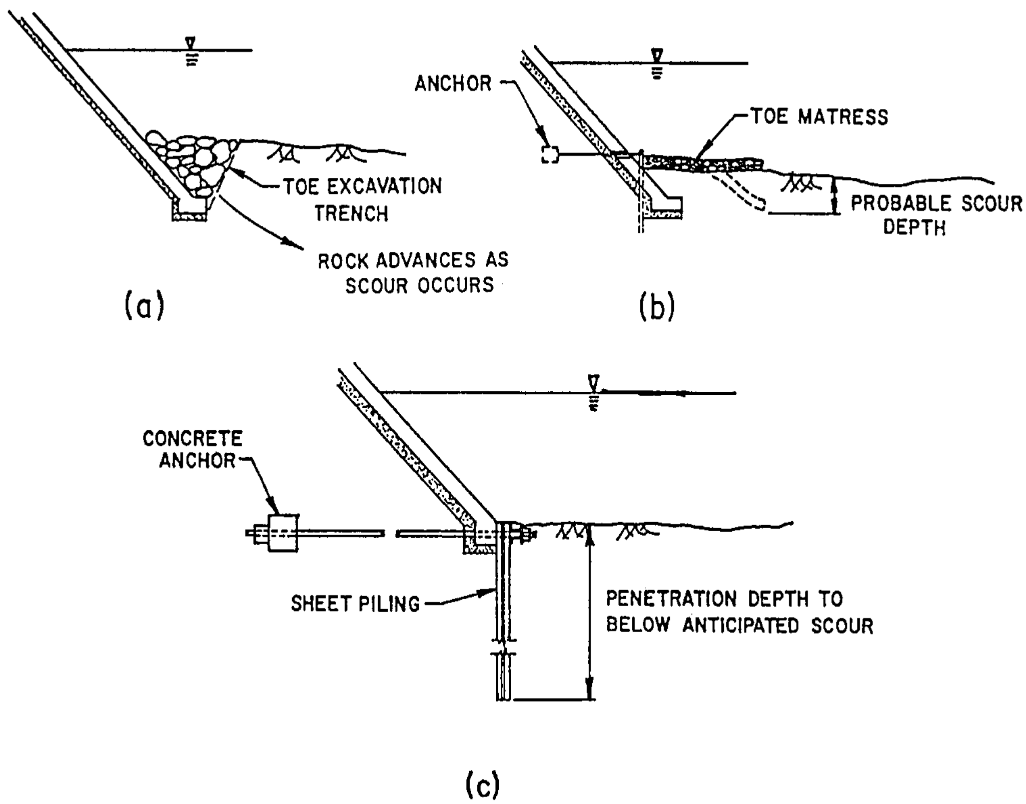


FIGURE 17-43 — Concrete Pavement Toe Details

17.7.5.1.8 Filter Design

Filters are required under all concrete pavement revetments to provide a zone of high permeability to carry off seepage water and prevent damage to the overlying structure from uplift pressures. A 4-in to 6-in granular filter is required beneath the pavement to provide an adequate drainage zone.

The filter can consist of well-graded granular material or uniformly graded granular material with an underlying filter fabric. The filter should be designed to provide a high degree of permeability while preventing base material particles from penetrating the filter, thus causing clogging and failure of the protective filter layer.

17.7.5.1.9 Pressure Relief

Weep holes should be provided in the revetment to relieve hydrostatic pressure buildup behind the pavement surface (see Figure 17-42). Weeps should extend through the pavement surface and into the granular underdrain or filter layer.

Weeps should consist of 3-in diameter pipes having a maximum horizontal spacing of 6 ft and a maximum vertical spacing of 10 ft. The buried end of the weep should be covered with wire screening or filter fabric of a gage that will prevent passage of the gravel filter layer.

Alternatively, a closed-end pipe with horizontal slits can be used for the drain; in this case, the slits must be of a size that will not pass the granular filter material.

17.7.5.2 Construction

Design details for concrete slope pavement are illustrated in Figure 17-42. The following construction procedures and specifications are given:

- Normal construction procedures include (a) bank clearing and grading; (b) development of a foundation; (c) trenching and setting forms for stubs; (d) placing the filter layer; (e) forming for and placing the concrete pavement (including any special adaptations necessary for the revetment toe); (f) backfilling toe trenches (if required); and (g) vegetation of disturbed areas.

- The usual specifications for placing and curing structural concrete should apply to concrete slope paving.

- Subgrade should be dampened before placement of the concrete.

- Reinforcement must be supported so that it will be maintained in its proper position in the completed paving.

- Slabs should be laid in horizontal courses, with cold joints without filler between courses. These joints should be formed with 3/4-in lumber, which should be removed and the joint left open upon completion.

- Vertical expansion joints should run normal to the bank at 15-ft to 30-ft intervals. These joints should be formed using joint filler.

- Headers or forms for use during screeding or rodding operations must be strong enough and so spaced that adjustment will not be necessary during placement operations.

17.7.6 Soil-Cement

Soil-cement is an acceptable method of slope protection for dams, dikes, levees, channels, coastal shorelines and highway embankments. Soil-cement can also be used to construct impervious cores in retention dam type structures and provide a protective facing. On most projects, soil-cement is constructed in stair-step fashion by placing and compacting the soil-cement in horizontal layers stair-stepped up the embankment (Figure 17-44). This facilitates placement using common highway construction equipment. Embankment slopes of from 1V:2.5H and 1V:4H and horizontal layer widths of from 7 ft to 9 ft provide minimum protective thicknesses of approximately 1.5 ft to 2.5 ft measured normal to the slope.

A wide variety of soils can be used to make durable soil-cement slope protection. The Portland Cement Association (PCA) has data on soil types, gradations, costs and testing procedures. The PCA also has data on placement and compaction methods.

Use of soil-cement does not require any unusual design considerations for the embankment. Proper embankment design procedures should be followed, based on individual project conditions, to prevent subsidence or any other type of embankment distress.

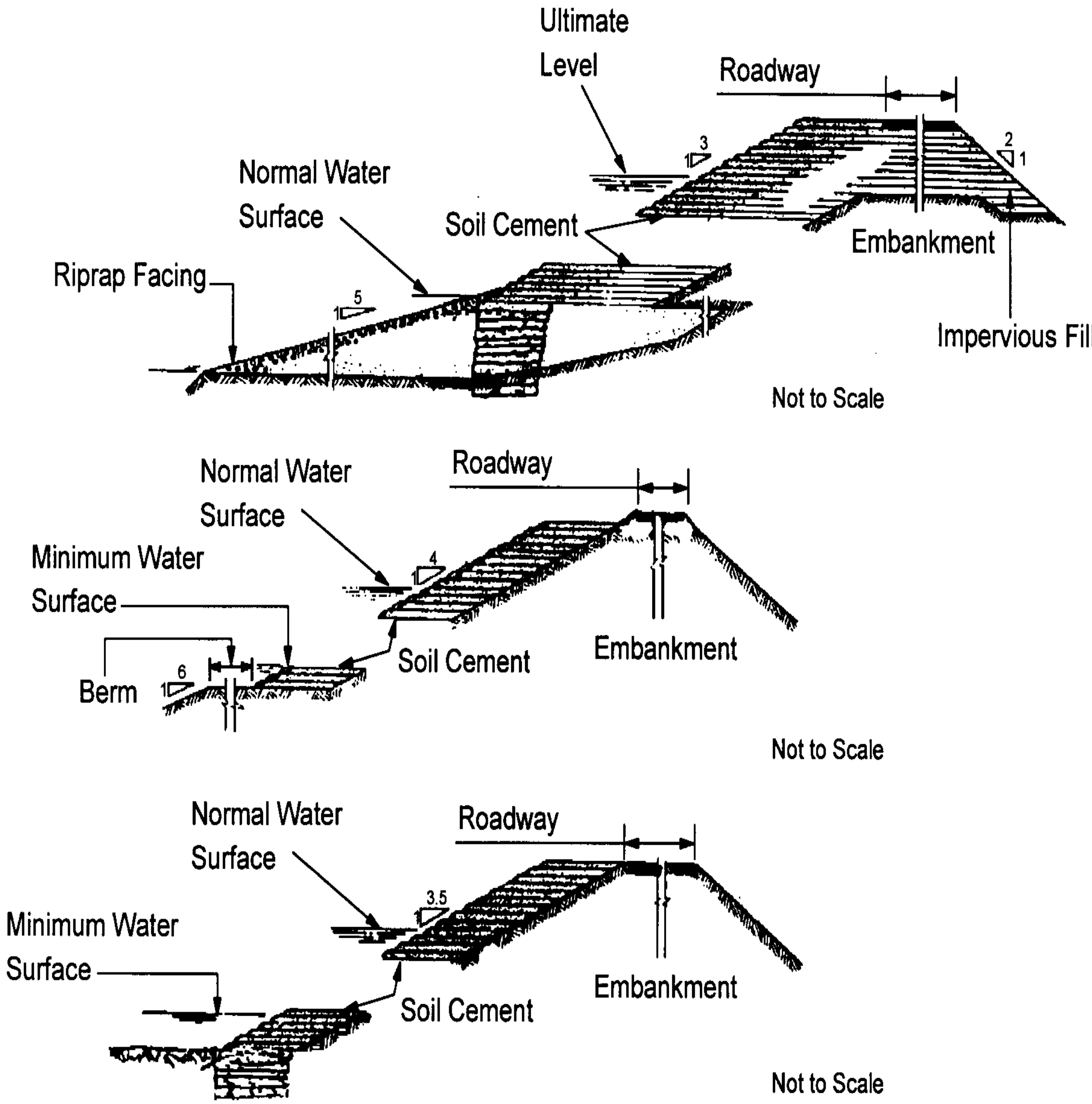


FIGURE 17-44 — Details and Dimensions of Three Soil-Cement Facings

Design Guidelines

17.7.6.1 Design Guidelines

17.7.6.1.1 Top, Toe and End Features

An important consideration in the design of soil-cement facing is to ensure that all extremities of the facing are tied into non-erodible sections or abutments. Adequate freeboard and carrying the soil-cement to the paved roadway, plus a lower-section detail as shown in Figure 17-44, will minimize erosion from behind the crest and under the toe of the facing. The ends of the facing should terminate smoothly in flat slopes or against rocky abutments. In some installations, a small amount of rock riprap may be placed over and adjacent to the edges of the soil-cement at its contact with the abutments.

Where miscellaneous structures (e.g., culverts) extend through the facing, the area immediately adjacent to such structures is constructed by placing and compacting the soil-cement by hand or with small equipment or by using a lean-mix concrete.

17.7.6.1.2 Special Conditions

Slope stability is provided to embankments by the strength and impermeability of the soil-cement facing. Special design considerations usually are not necessary in soil-cement-faced embankments. It is necessary to utilize proper design and analysis procedures to ensure the structural and hydraulic integrity of the embankment. Conditions most commonly requiring special analysis include subsidence of the embankment or rapid drawdown of the reservoir or river.

17.7.6.1.3 Subsidence

Embankment subsidence results from a compressible foundation, settlement within the embankment itself or both. Analyzing the possible effects of such a condition involves a number of assumptions by the designer concerning the embankment behavior. Combining these assumptions with the characteristics of the facing, a structural analysis of the condition can be made. If the unit weight and flexural strength of the soil-cement are not known, they can be taken as 120 lb/ft^3 and 150 - 200 lb/in^2, respectively. The layer effect can usually be ignored.

Note: The post-construction appearance of a pattern of narrow surface cracks approximately 10 ft to 20 ft apart is evidence of normal hardening of the soil-cement. Substantial embankment subsidence conceivably could allow the facing to settle back in large sections coinciding with the normal shrinkage crack pattern. If such settlement of the soil-cement with separation at the shrinkable cracks occurs, the slope remains adequately protected unless the settlement is large enough to allow the outer face of a settling section to move past the inner face of an adjoining section.

17.7.6.1.4 Rapid Drawdown

Rapid drawdown exceeding 15 ft or more within a few days theoretically produces hydrostatic pressure from moisture trapped in the embankment against the back of the facing. Three design concepts that may be used to prevent damage due to rapid drawdown-induced pressure are:

1. designing the embankment so that its least permeable zone is immediately adjacent to the soil-cement facing, which ensures that seepage through cracks in the facing will not build up a pool of water sufficient to produce damaging hydrostatic pressure;

2. designing sufficient facing weight to resist any uplift pressures that may develop; and

3. providing free drainage behind, through or under the soil-cement facing to prevent adverse hydrostatic pressure.

17.7.6.2 Construction

The method of construction (central plant or mixed in place) should be considered by the designer in determining the facing cross section. Both methods have been successfully used for soil-cement slope protection. The central plant method, however, allows faster production and provides maximum control of mixing operations. With the mixed-in-place method, mixing should be deep enough so that there will be no unmixed seams between the layers, but excessive striking of the soil-cement below the layer being mixed should be avoided. A compacted layer thickness of 6 in is most widely used, with the recommended maximum being 9 in for efficient, uniform compaction.

The central-plant method should be more economical for all but the smallest projects, but it is well to allow the contractor the option of using either method where the quantity of soil-cement involved is only a few thousand cubic feet. The PCA has sample specifications regarding these two construction methods.

17.7.7 Bendway Weirs

Bendway weirs, also referred to as stream barbs, bank barbs and reverse sills, are low-height stone sills used to improve lateral stream stability and flow alignment problems at river bends and highway crossings. Bendway weirs are used for bankline protection and flow alignment in meandering channel bends.

Bendway weirs are similar in appearance to stone spurs but function in an entirely different manner. Spurs are designed so that the flow is diverted around or through the spur. Bendway weirs are low structures, visible only at low flow, designed to redirect flow normal to the axis of the weir. The weirs consist of a number of low riprap walls extending out into the channel from the bank, spaced through the bend. Bendway weirs are keyed into the banks to prevent flow from flanking the end of the weir. Initial installation and design procedures based on some limited research are described in USACE *(10)* and NRCS *(11)*. Most current guidance can be found in Reference *(2)*.

17.7.7.1 Height of Weir

The height of the weir is determined by analyzing flow conditions at the site. The criteria for determining height to produce the desired results are based on the following:

- Height should be 30% to 50% of the mean annual flood depth.

- It should be below normal or seasonal mean water level.

- It should be at or above mean low-water level.

- If the channel is used for boat traffic it should be low enough to allow boats to pass over the weir safely.

These criteria are shown in Figure 17-45.

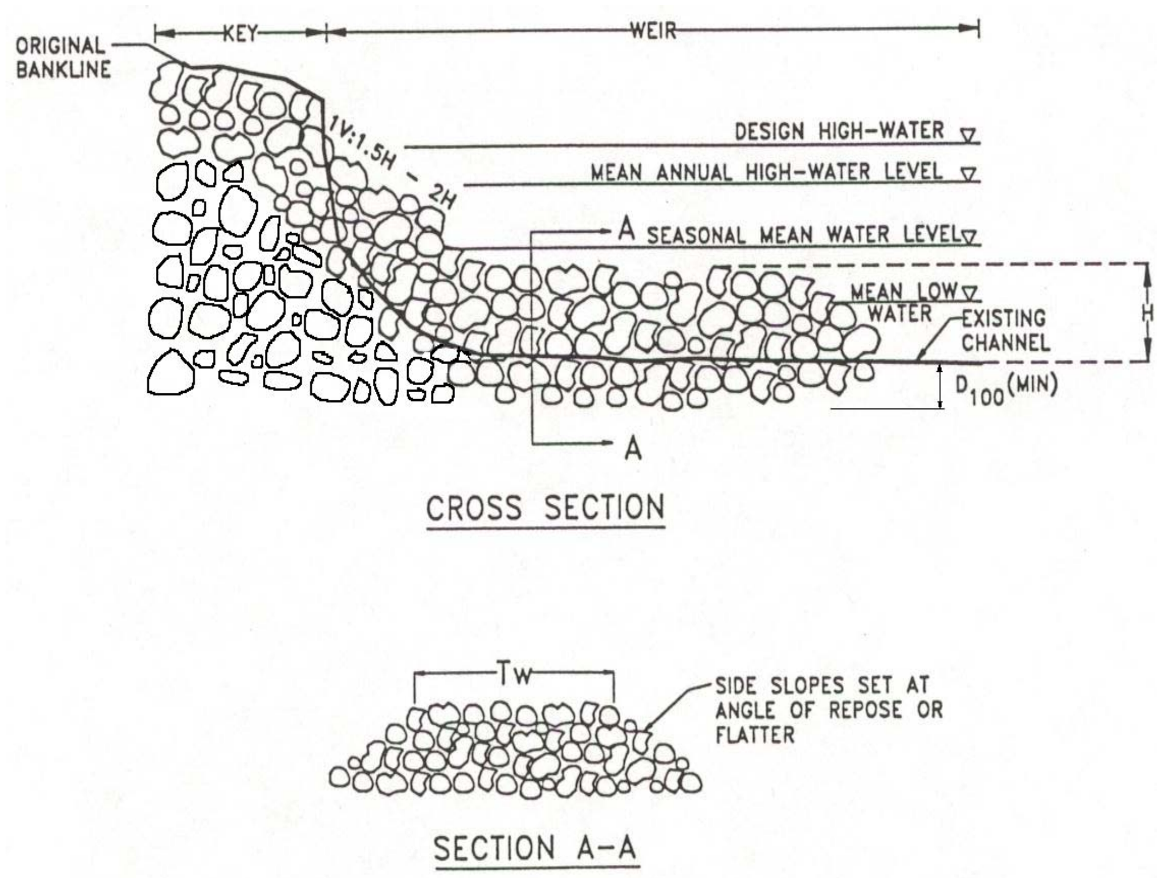


Figure 17-45 — Bendway Typical Section

17.7.7.2 Angle of Projection

The angle of projection (θ) of the weir into the stream is based on the location of the weir in the bend and the upstream bankline tangent. For convenience, the angle can be measured to the chord shown on Figure 17-46. Ideally, the angle of attack of the flow lines at high-water flow should be no greater than 30° and, at low flow, the angle of attack of the flow lines should be no less than 15° to the normal of the centerline of the first few weirs. If the low-flow angle of approach is head-on, the weir will be ineffective and serve to divide the flow. Bank scalloping will result. If the angle of the approach flow line of high-water flow is too large, the weir will not be able to effectively redirect the flow to the desired direction. The resulting projection angle of the weir should be between 50° and 85° to the chord defined by the intersection of the centerline of the weir and the centerline of the adjacent weir upstream with the bank line (see Figure 17-46). Observations by LaGrone *(10)* indicate that the angle, θ, is most important in redirecting flow. The upstream edge of the weir should be a well-defined straight line.

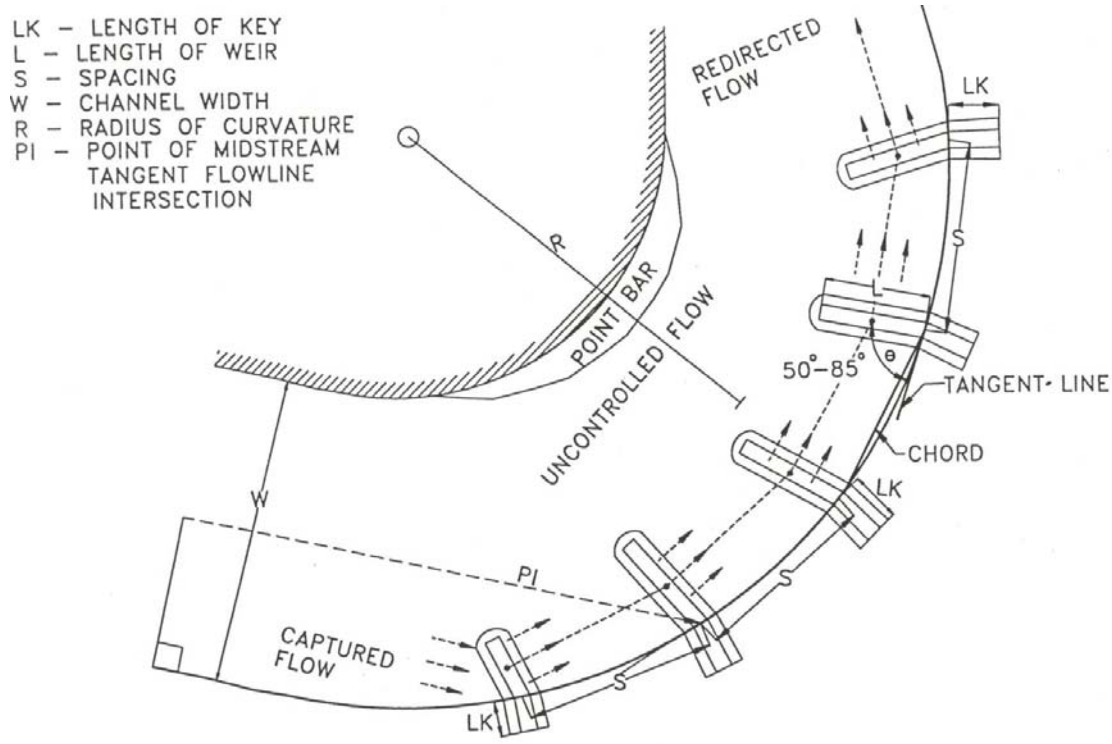


FIGURE 17-46 — Bendway Weir Typical Plan View

17.7.7.3 Length of Weir

The length of the weir does not have a definitive design, but the following criteria are offered as guidance. Primary guidance for the length of weirs designed for bank protection is that they need not exceed one fourth of the channel width. The maximum length should not exceed one third of the channel width. Weirs this long can cause the flow pattern to impact the opposite bank. Additional guidance for weir length is that it should be long enough to extend across the thalweg. A length equal to 1.5 to 2.0 times the distance from the bank to the thalweg has proven satisfactory. The length of the weir will define the spacing between weirs.

17.7.7.4 Weir Location and Spacing

A first short weir should be located a distance (S) upstream from the point where a projection (PI) of the mid-channel flow line of the approach flow would impact the bank (see Figure 17-46). Additional weirs are then located at equal distances (S), based on site conditions and using sound judgment. The spacing (S) of the weirs is dependent on the streamflow leaving the weir and its impact point on the bank. The following formulas are given for guidance in establishing the spacing:

$$S = \left(\frac{R}{W}\right)^{0.8}\left(\frac{L}{W}\right)^{0.3} \quad \text{(LaGrone } \textit{(10)}\text{)}$$

$$S = (4 \text{ to } 5)\,L \qquad \text{(Seale } \textit{(12)}\text{)}$$

where: R = Channel radius of curvature
W = Channel width
L = Weir length

Maximum spacing (S_{max}) is given by: $S_{max} = R\sqrt{1-\left(1-\frac{L}{R}\right)^2}$

Maximum spacing is not recommended but, in cases where some erosion is permissible, the spacing may be set between S and S_{max}. The spacing and weir length, given by the formulas, should be investigated to determine if they will direct the flow as desired. Streamlines approaching and leaving the weirs should be analyzed and drawn in plan form. A suggested method for evaluating the spacing of the weirs during construction is to construct the upstream weir first. Then adjust the location of the next downstream weir based on where the resulting flow hits the stream bank. Continue this process with each weir in turn through the last weir.

17.7.7.5 Length of Key

To prevent flanking flow, bendway weirs should be keyed into the bank. The key length (LK) is based on a 20° flow expansion angle (Figure 17-47). Two guidance formulas for setting key length are given, based on the radius of curvature of the channel:

For a large radius of curvature, $R > 5W$ and $S > \frac{L}{\tan(20°)}$

$$LK = S\tan(20°) - L \quad \text{(LaGrone } \textit{(10)}\text{)}$$

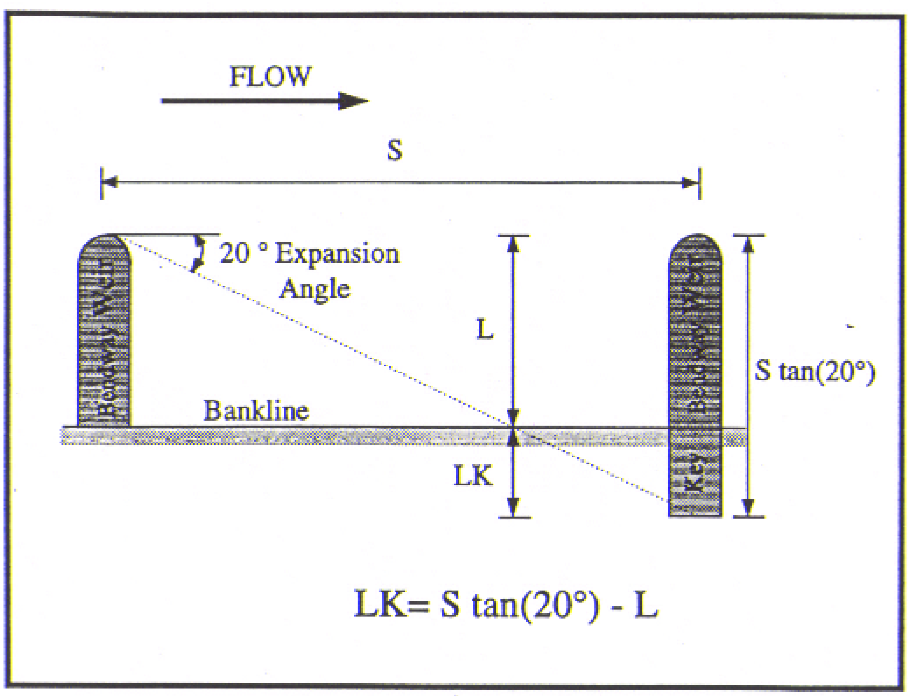


FIGURE 17-47 — Length of Key for Mild Bends

For small radius of curvature, $R < 5W$ and $S < \dfrac{L}{\tan(20^\circ)}$ tan 20° = 0.36397

$$LK = \frac{L}{2}\left(\frac{W}{L}\right)^{0.3}\left(\frac{S}{R}\right)^{0.5}$$

LK should not be less than 1.5 times the total bank height. NRCS guidelines for barbs *(12)* and short weirs specify that the key should be 8 ft or $4(D_{50})$, whichever is greater.

The key may be in line with the weir as shown in Figure 17-47 or perpendicular to the tangent at the bank as shown in Figure 17-46. The depth of the key should extend down to the depth of the thalweg for the full length into the bank.

17.7.7.6 Top Width

The top width (T_w) of the weir may vary between 3 ft to 12 ft but should be no less than 2 to 3 times D_{100}. Where the length of the weir is so long that it cannot be constructed from the bank, the weir must be built from a barge or made wide enough for construction equipment to operate on top of the weir.

17.7.7.7 Filter

The design for a filter between the riprap in the weir and the soil should be determined utilizing the filter design procedure in Section 17.7.1.5.

17.7.7.8 Number of Weirs

The number of weirs should be sufficient to accomplish the task of stabilizing the bank. The weir length and spacing can be adjusted to minimize the number of weirs. Usually, a minimum of three weirs should be used.

17.7.7.9 Construction

Construction of the weir normally should be done during low-flow periods. Construction procedures will vary due to the size of the channel. As indicated above, construction in large rivers may have to be performed from a barge or from the top of the weir itself.

17.7.7.10 Material Specifications

The following applies to weir materials:

1. Stone should be angular. Not more than 30% of the stone should have a length exceeding 2.5 times its thickness.

2. No stone should be longer than 3.5 times its thickness.

3. Stone should be well graded but with only a limited amount of material less than half the median stone size. Because the stone often will be placed in moving water, the smaller stone will be subject to displacement by the flow during installation.

4. Construction material should be quarry-run stone or broken, clean concrete. High-quality material is recommended for long-term performance.

5. Material sizing should be based on standard riprap sizing formulas for turbulent flow. Typically, the size should be approximately 20% greater than that computed from non-turbulent riprap sizing formulas. The riprap D_{50} typically ranges between 1 ft to 3 ft and should be in the 100-lb to 1,000-lb range. The D_{100} rock size should be at least 3 times the calculated D_{50} size. The minimum rock size should be not less than the D_{100} of the streambed material.

17.7.8 Vegetative Plantings and Soil Bioengineering Systems

The Department has on going research on appropriate use of vegetative plantings and soil bioengineering. For guidance on these systems contact UDOT Central Hydraulics, for information and reference see reference *(11)*.

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(4) Federal Highway Administration, *Geosynthetic Design and Construction Guidelines*, FHWA-HI-95-038, 1995, revised 1998.

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